AD 738547

Technical Report *October 1971*

BLAST RESPONSE OF FIVE NFSS BUILDINGS

Prepared for:

OFFICE OF CIVIL DEFENSE
OFFICE OF THE SECRETARY OF THE ARMY
WASHINGTON, D.C. 20310

CONTRACT DAHC20-71-C-0292
OCD Work Unit 1154I

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STANFORD RESEARCH INSTITUTE
Menlo Park, California 94025 · U.S.A.

86

UNCLASSIFIED
Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY *(Corporate author)* STANFORD RESEARCH INSTITUTE Menlo Park, California 94025	2a. REPORT SECURITY CLASSIFICATION Unclassified 2b. GROUP Not applicable

3. REPORT TITLE

BLAST RESPONSE OF FIVE NFSS BUILDINGS

4. DESCRIPTIVE NOTES *(Type of report and inclusive dates)*

Technical Report

5. AUTHOR(S) *(First name, middle initial, last name)*

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6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
October 1971	110	5

8a. CONTRACT OR GRANT NO. OCD-DAHC20-71-C-0292 b. PROJECT NO. Work Unit No. 1154I c. d.	9a. ORIGINATOR'S REPORT NUMBER(S) 9b. OTHER REPORT NO(S) *(Any other numbers that may be assigned this report)*

10. DISTRIBUTION STATEMENT

Approved for public release; distribution unlimited.

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY Office of Civil Defense Office of the Secretary of the Army Washington, D.C. 20310

13. ABSTRACT

The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Past efforts in this program have been concerned with examining exterior walls, window glass, and steel frame connections. In this phase, the previously developed mathematical models for exterior walls were used to predict the collapse overpressure for selected structures. The report presents the results of the dynamic analysis of the exterior walls of five structures located in the Greensboro-High Point SMSA of North Carolina.

DD FORM 1 NOV 65 1473 REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

UNCLASSIFIED
Security Classification

14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Walls						
Structures						
Blast Shelters						
Analysis						
Blast						
Mechanics						

Technical Report

Detachable Summary

October 1971

BLAST RESPONSE OF FIVE NFSS BUILDINGS

By: C. K. WIEHLE
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Facilities and Housing Research

Prepared for:

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CONTRACT DAHC20-71-C-0292
OCD Work Unit 1154I

SRI Project 1219

OCD REVIEW NOTICE

SUMMARY

Objective

The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Background

Past efforts in this program have been concerned with examining exterior walls, window glass, steel frame connections, and applications to actual structures. This report presents the results of the dynamic analysis of the exterior walls of five structures located in the Greensboro-High Point Standard Metropolitan Statistical Area (SMSA) of North Carolina.

As part of an integrated program to develop a survey procedure for all nuclear weapon effects, Research Triangle Institute (RTI) made an initial on-site field survey during November 1970 of five preselected NFSS buildings in Detroit, Michigan. A complete copy of the survey information and of the building plans was provided to SRI for analysis of the buildings. The results of the dynamic analysis of the five Detroit buildings were presented in a previous report.*

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* Wiehle, C. K., and J. L. Bockholt, Existing Structures Evaluation, Part V: Applications, Stanford Research Institute (for Office of Civil Defense), Menlo Park, California, July 1971.

To provide additional input information for the development of the all-effects survey, RTI made a second on-site field survey in July 1971 of five buildings located in the vicinity of Greensboro, North Carolina. In a manner similar to that employed in the analysis of the Detroit buildings, SRI made a dynamic analysis of each of the Greensboro buildings.

At the present time the evaluation procedure has not been extended to include the collapse of the structural frame under dynamic loading. Therefore, to use the interim techniques for predicting the collapse of the exterior walls, it was necessary to assume that the frame did not fail at a lower overpressure level than the exterior walls. For four of the Greensboro buildings this assumption probably did not significantly influence the collapse predictions. However, as discussed in the main body of the report, it is most probable that an overall collapse of the frame of one of the Greensboro buildings would occur at a lower overpressure than that predicted for the exterior wall.

Analysis

The predicted collapse overpressures for all five Greensboro buildings and for both the field survey and building plan analyses are summarized in Table S-1. A comparison of the results of the analyses demonstrates that, when the proper building information is obtained in an on-site field survey, there is then generally good agreement between the collapse predictions made with both the field survey and building plan data. On the other hand, if certain construction details are not documented correctly, then the predictions from the two sets of data can vary by a wide degree.

The study of the five Greensboro buildings indicated that differences between comparative analyses, performed with survey and building plan data, varied by factors as great as nine. As noted in the discussion

Table S-1

SUMMARY OF WALL ANALYSES

Case*	Location: Side	Location: Story	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
Southern Furniture Exhibition Building								
IF1	ABD	2-10	A One-way	13	1.6	0.6	0.9	2.4
IP1	ABD	2-7, 9,10	A One-way	10	0.3	0.1	0.2	0.5
IP2	ABD	8	A One-way	10	Neglibible			
Greensboro Public Library								
IIF1	AB	1	A Two-way	12	6.0	1.5	4.1	7.9
IIF2	AB	2	A Two-way	12	6.9	1.2	5.4	8.5
IIF3	C	2	A Two-way	12	5.2	1.1	3.8	6.7
IIF2′	AB	2	A Two-way	8	2.7	1.5	0.8	4.6
IIP1	AB	1	A Two-way	8	5.2	1.8	2.8	7.5
IIP2	A	2	A Two-way	8	5.1	1.5	3.2	7.0
IIP3	C	2	A Two-way	12	5.1	1.3	3.4	6.8
IIP4	B	2	A Two-way	8	5.6	1.5	3.7	7.4
IIP2′	A	2	A Two-way	8	3.5	2.3	0.6	6.4

Table S-1 (continued)

Case*	Location Side	Location Story	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
Laura Cone Dormitory								
IIIF1	AC	2-9	A Two-way	6	7.6	2.0	5.0	10.1
IIIF2	BD	2-9	A Two-way	6	5.4	0.8	4.3	6.4
IIIF1′	AC	2-9	A One-way	6	2.8	0.8	1.7	3.8
IIIP1	AC	2-9	U-3	4	1.0	0.1	0.9	1.1
IIIP2	BD	2-9	A One-way	12	11.2	1.1	9.7	12.6
IIIP1′	AC	2-9	U-1	6	0.7	0.3	0.3	1.0
Willa B. Player Hall								
IVF1	A	1	U-2	16	7.7	0.7	6.9	8.6
IVF2	BC	1	U-2	16	8.3	0.6	7.5	9.1
IVF3	A	2	U-2	12	4.9	0.5	4.2	5.5
IVF4	BCD	2	U-2	12	5.2	0.3	4.8	5.7
IVF5	A	3	U-2	12	3.7	0.2	3.4	4.0
IVF6	BCD	3	U-2	12	3.7	0.2	3.4	3.9
IVF2′	ABCD	1	U-1	8	1.9	0.1	1.8	2.0
IVF4′	ABCD	2	U-1	8	1.3	0.03	1.24	1.32
IVF6′	ABCD	3	U-1	8	0.5	0.04	0.43	0.52
IVP1	A	1	U-2	16	7.7	0.7	6.7	8.6
IVP2	BC	1	U-2	16	8.3	0.7	7.4	9.3
IVP3	A	2	U-2	12	4.6	0.5	4.0	5.2
IVP4	BCD	2	U-2	12	5.0	0.5	4.4	5.6
IVP5	A	3	U-2	12	3.1	0.2	2.8	3.3
IVP6	BCD	3	U-2	12	3.3	0.1	3.1	3.5
IVP3′	A	1-3	U-1	8	0.2	0.1	0.1	0.3
IVP4′	ABCD	1-3	A One-way	8	4.6	2.4	1.5	7.7

Table S-1 (concluded)

Case*	Location: Side	Location: Story	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
North Carolina National Bank								
VF1	A	1	A One-way	13	3.9	0.7	3.0	4.8
VF2	B	1	A One-way	13	1.8	0.2	1.6	2.0
VF3	ABCD	2-8	A One-way	13	12.4	2.6	9.0	15.7
VP1	A	1	A One-way	17	16.4	4.2	11.0	21.8
VP2	B	1	A One-way	17	5.4	0.7	4.6	6.3
VP3	ABCD	2-8	A Two-way	13	15.7	4.0	10.5	20.8

* The prefix F identifies walls analyzed using field survey data, and P those analyzed using building plan data. The prime identifies interior partitions.

† Each wall is designated with a letter to identify the wall type and a number to identify the wall support condition. The key to the wall types and support cases are given in Table S-2.

Table S-2

WALL TYPE AND SUPPORT KEY

Letter	Wall Type
U	Unreinforced masonry unit wall
A	Arching wall
RC	Reinforced concrete wall

Number	Support Case
1	Two-way, simply supported on four edges
2	Two-way, fixed on four edges
3	Two-way, fixed on vertical edges and simply supported on horizontal edges
4	Two-way, simply supported on vertical edges and fixed on horizontal edges
5	One-way, simply supported on opposite edges
6	One-way, fixed on opposite edges
7	One-way, propped cantilever
8	One-way, cantilever

of each building in the body of the report, the difference in collapse overpressure of a specific wall, using the field survey or building plan data, can be attributed primarily to the difference in the assumed support conditions. A contributing factor was the variation in the wall thickness obtained from the survey and plan information.

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ABSTRACT

The objective of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Past efforts in this program have been concerned with examining exterior walls, window glass, and steel frame connections. In this phase, the previously developed mathematical models for exterior walls were used to predict the collapse overpressure for selected structures. The report presents the results of the dynamic analysis of the exterior walls of five structures located in the Greensboro-High Point SMSA of North Carolina.

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I INTRODUCTION

Under contract to the Office of Civil Defense, Stanford Research Institute is developing a procedure for the evaluation of existing structures subjected to nuclear air blast. The objectives of the overall research program is to develop an evaluation procedure for determining the blast protection afforded by existing NFSS-type structures and private residences. The purpose of the application phase of the research presented in this report was to use the interim evaluation technique to predict the damage to actual NFSS structures.

Background

Past efforts in this program have been concerned with examining exterior walls (Refs. 1 and 2), window glass (Ref. 3), steel frame connections (Ref. 4), and applications (Ref. 5). This report presents the results of the dynamic analysis of the exterior walls of five structures located in the Greensboro-High Point SMSA of North Carolina.

As part of an integrated program to develop a survey procedure for all nuclear weapon effects, Research Triangle Institute (RTI) made an initial on-site field survey during November 1970 of five preselected NFSS buildings in Detroit, Michigan. The survey was conducted primarily to obtain a complete structural description of the buildings that would be adequate for building damage and casualty prediction purposes. The results of the field survey were recorded on predesigned forms and included sketches and photographs. A complete copy of this information, together with the building plans, was provided to SRI for analysis of the buildings. The results of the dynamic analysis of the five Detroit buildings were presented in Ref. 5.

To provide additional input information for the development of the all-effects survey, RTI made a second on-site field survey in July 1971 of five buildings located in the vicinity of Greensboro, North Carolina. In a manner similar to that employed in the analysis of the Detroit buildings presented in Ref. 5, two dynamic analyses were made of each of the Greensboro buildings in this study. The first analysis was made using the data obtained during the RTI on-site survey. A second analysis of the same building was then made independently using data obtained from the actual building plans. This procedure provided a check on the adequacy of the survey technique and the proposed field survey data form and emphasized areas of possible improvement.

It should be noted that when a discrepancy occurred between the survey and plan data during this study, no attempt was made to determine which was correct. On the one hand, it is possible that the building plans could be in error, since construction drawings do not necessarily reflect the as-built condition of a structure. On the other hand, however, the survey data could be in error, since some parameters, e.g., the existence or width of a wall cavity, are difficult to determine by on-site inspection without considerable effort or special equipment. Because the accuracy of the data can be important to the building collapse predictions, consideration should be given in any future survey and blast analysis exercise to a resurvey of important parameters when a discrepancy occurs between the survey and plan data.

Analysis Limitations and Discussion

The predictions of the collapse overpressure of the buildings were based on a dynamic analysis of the exterior walls using the procedures presented in Refs. 1 and 2. That is, the intent in this study was to predict the blast damage to actual NFSS structures, even though only interim techniques were available for analyzing wall elements. This pro-

cedure was of value in providing guidance in planning the research effort and in providing interim predictions for the collapse overpressure of actual structures for use by the Office of Civil Defense (OCD).

At the present time the evaluation procedure has not been extended to include the collapse of the structural frame under dynamic loading. Therefore, to use the interim techniques for predicting the collapse of the exterior walls, it was necessary to assume that the frame did not fail at a lower overpressure level than the exterior walls. For four of the Greensboro buildings this assumption probably did not significantly influence the collapse predictions. However, as was noted for two of the structures in the Detroit study in Ref. 5, it is most probable that an overall collapse of the frame of one of the Greensboro buildings (i.e., the North Carolina National Bank) would occur at a lower overpressure than that predicted for the exterior wall.

The collapse of the floor slab over basement areas is an important consideration in determining the survivors in nuclear blast environments. However, collapse predictions for the floors in the Greensboro-High Point buildings could not be included in this effort because the procedures are currently being developed. The analysis of floor slabs will be included in the building collapse predictions when the procedures become available.

In addition, the method of construction of an arching-type wall is extremely important in the determination of its resistance function. For example, if a wall is constructed so that the closing joint at the top of the wall (between the wall and the floor beam or slab) is well mortared, it is reasonable to assume that the wall can develop its maximum arching force. On the other hand, if the top mortar joint is improperly made or if a gap exists between the wall and beam, the arching resistance is reduced in proportion to the size of the gap.* Also, a

* The resistance function for arching walls with a gap or elastic supports is presented in Ref. 1.

gap or improperly mortared top joint may result in a collapse mechanism that prevents the development of arching resistance. Since there is no information available on the actual construction techniques used for any of the structures analyzed in this study, it was assumed that if the wall was of the arching type, the maximum arching resistance was developed.

For the evaluation of the exterior wall elements in this study, failure implies collapse or disintegration of the wall. Furthermore, the predicted collapse overpressures given are for the incipient collapse of the wall, which is defined as that point in the response where the wall can be considered as on the threshold of collapse. The pressure at incipient collapse is therefore the load that is just sufficient in magnitude to cause a collapse of the wall--a load of slightly lesser magnitude would not result in collapse.

It should be noted that the load-time function on a wall in an actual structure subjected to nuclear blast is a complex phenomenon, and a precise description of the loading function is not too meaningful in comparing collapse predictions. Therefore, the predicted collapse overpressures given in this report are the peak incident overpressures of the free-field blast wave that result in collapse of the wall.

Acknowledgments

The authors gratefully acknowledge the assistance and guidance of G. N. Sisson and N. A. Meador of the Office of Civil Defense during the conduct of this program. The authors are also grateful to M. D. Wright of Research Triangle Institute for providing the photographs and field survey information for the Greensboro-High Point buildings.

II BUILDING ANALYSIS--GREENSBORO-HIGH POINT

Introduction

The analysis of each of the five Greensboro-High Point NFSS buildings is presented in this section. In each subsection a description of the building is given, together with a copy of the photographs provided by RTI. The building is described as it was designed, and therefore there may be some discrepancies between the building descriptions and the field survey data presented in the Appendix. Following the description, the analysis of the building is presented in two subsections, the first using the field survey data and the second using the building plan data.

The exterior walls for which collapse predictions were made were analyzed using the probability technique presented in Ref. 2. Therefore, the collapse values are given as having a 10-, 50-, or 90-percent probability of occurrence.

In general, the procedure used to make the collapse predictions was first to make a detailed examination of the field survey data, sketches, and photographs. From this information the walls that were believed to be important to the failure of the structure or to the production of significant casualties were selected for analysis. Although it was not feasible to analyze every wall in all five buildings for this phase of the effort, the walls selected were representative for each building. The input data required in the computer programs consist of the wall and load properties, including probability distributions where needed. Although the geometric wall properties were usually available from the field survey data, the properties of the masonry materials were not available. Since this is generally the case for existing structures,

it was necessary to assume values for the material properties required in the analysis. The material properties used in this study are summarized in Table 1; they were based on previous data.

After the walls were analyzed using the field survey data, the building plans were examined in detail and a new set of input data was prepared for each building. The properties of the masonry materials were usually not specified on the plans, and therefore the values in Table 1 were also used for the building plan data analysis.

An important factor in the prediction of the collapse of a structure is the method used to determine the transient blast loading. For this study the front face, interior, and net loading on each wall was calculated by the procedure discussed in Ref. 2. It was assumed that each wall being analyzed was struck at normal incidence by a plane Mach waveform created by a 1 Mt surface burst: that is, each wall was analyzed as though it were the "front face" of the building with an ideal blast wave advancing at normal incidence to it. For this limited study it was not possible to analyze the side and rear walls for the effect of a blast wave engulfing the structure. As noted in Ref. 2, because of the time relationship between the interior and exterior blast pressures and the design of some wall elements, it is possible that a side or rear wall of a structure may be expected to collapse at a lower incident overpressure than that predicted for the front wall.

Southern Furniture Exhibition Building

Description

The Southern Furniture Exhibition building constructed in 1967, is located on East Green Drive, in High Point, North Carolina. The building consists of 11 stories with a lower level and basement below the first-floor level. The overall height of the building is 153 ft and plan dimensions of 145 ft by 233 ft provide an area of about

Table 1

STRUCTURAL PROPERTIES OF MASONRY MATERIALS

Material	γ (pcf)	f_r (psi) Mean	f_r (psi) Standard Deviation	f'_m (psi) Mean	f'_m (psi) Standard Deviation	E_m* (psi)	t_f (in.)
Brick	120	100	42	2000	600	$1.0\text{x}10^6$	--
Concrete	145	$8\sqrt{f'_{dc}}$	0	3750	0	$57619\sqrt{f'_c}$	--
Concrete block, 4 in.	90	60	25	1200	350	$1.0\text{x}10^6$	1.375
Concrete block, 8 in.	83	60	25	1200	350	$1.0\text{x}10^6$	1.500
Concrete block,12 in.	80	60	25	1200	350	$1.0\text{x}10^6$	1.750
Structural clay tile, 4 in.	75	50	20	1750	450	$.75\text{x}10^6$	0.75
Structural clay tile, 6 in.	60	50	20	1750	450	$.75\text{x}10^6$	0.75
Structural clay tile, 8 in.	50	50	20	1750	450	$.75\text{x}10^6$	0.75

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* Values given are for analyzing an unreinforced masonry wall without arching. For walls in which arching occurs, a mean value of $E_m = 1000 f'_m$ and a standard deviation of $300 f'_m$ are to be used.

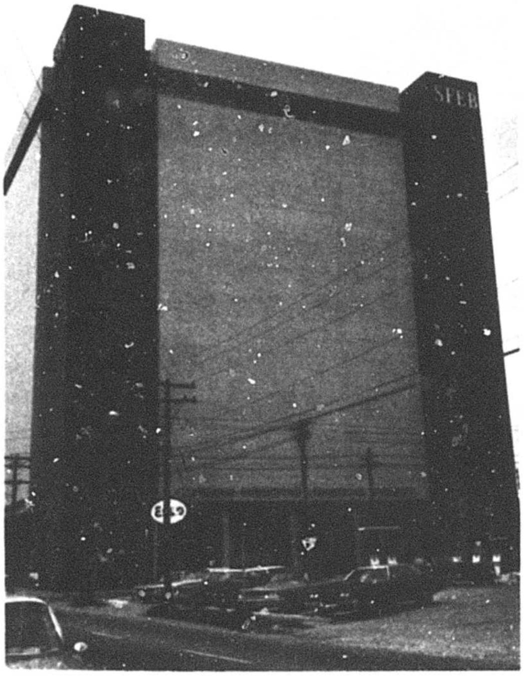
28,000 sq ft on the first floor and 31,000 sq ft on the upper floors. As noted on Figure 1, the building is mostly windowless except for the large glass areas on the first and eleventh stories. The exhibition building was constructed as a wing of an existing building for the full height on side C.

The frame is of structural steel and reinforced concrete composite-type construction. The floors on the first and lower level consist of reinforced concrete beams and one-way concrete joists with a 4-1/2-in. thick reinforced concrete slab. The upper floors are constructed with structural steel beams between columns and open-web steel joists that span between the beams and support a 4-in. thick concrete slab. Above the first floor level the floor extends 8 ft beyond the column lines.

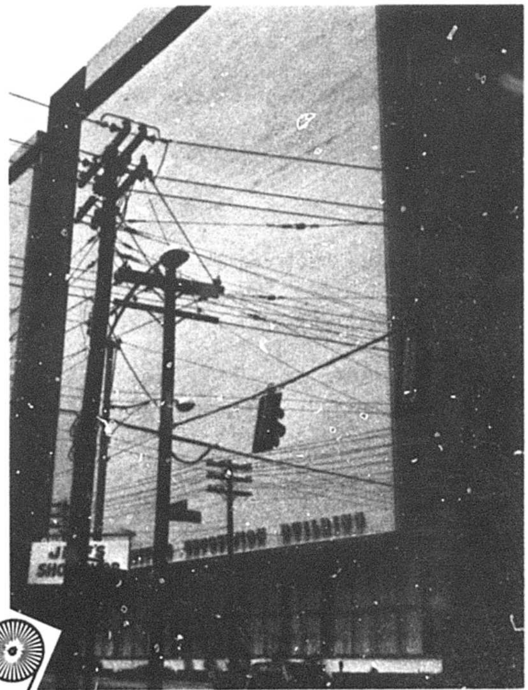
The exterior walls are constructed of a 4-in. thick brick facing with a 4-in. thick concrete block backing wythe and a 2-in. cavity. The walls are unreinforced and the 4-in. thick concrete block is inset between the floors; the brick facing is continuous over the floors. The interior partitions on the first story consist of either timber studwall or 8-in. thick nonload-bearing, concrete block construction. The upper stories contain very few permanent-type interior partitions except around the stair and utility areas.

Analysis

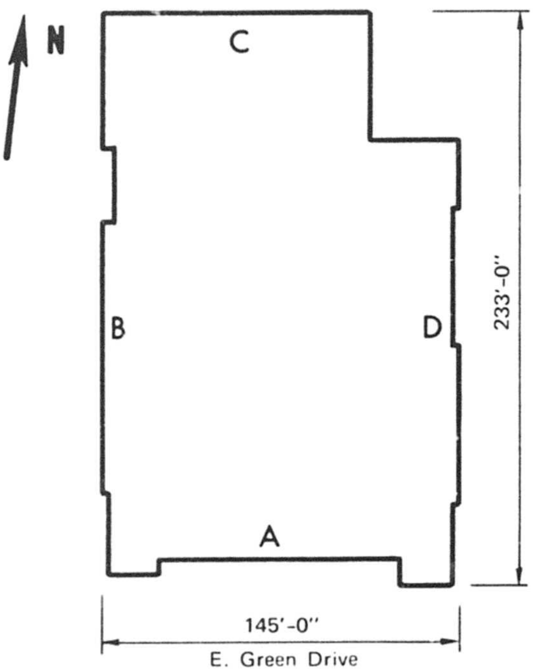
Field Survey Data. During the on-site survey the exterior walls of the Exhibition Building were classified as nonreinforced concrete block panel walls with brick masonry veneer. The 8-in. thick concrete block was described as inset in the frame and the 4-in. thick brick as continuous over the frame with a 1-in. cavity between the two wythes. Although the wall panel width was given in the data as 21 ft, the wall was analyzed as capable of developing only one-way structural action in the vertical direction, since the outer column line was shown on a sketch as located well inside the plane of the exterior wall.

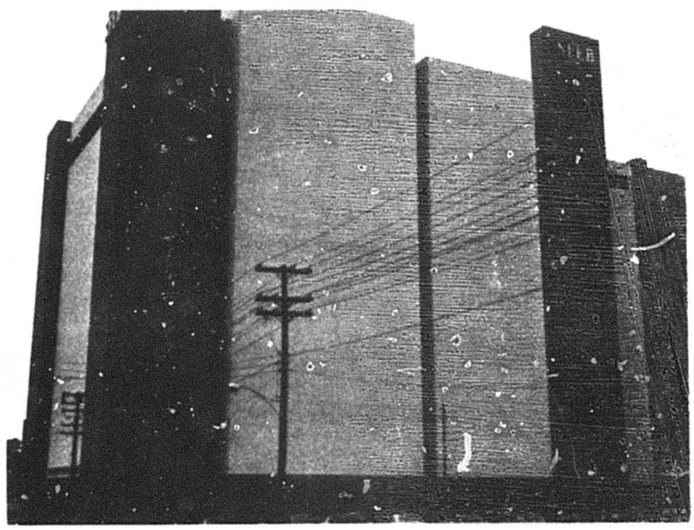


SIDE A



SIDE B





SIDE D

SOURCE: RTI.

FIGURE 1 PHOTOGRAPHS AND PLOT PLAN OF SOUTHERN FURNITURE EXPOSITION BUILDING

Since the Exhibition Building was constructed as an extension of an existing structure, there was no exterior wall on side C. Also, the walls on stories 1 and 11 were not analyzed since their resistance was controlled by the strength of the large windows. For sides A, B, and D and stories 2 through 10, the walls were analyzed as one-way arching walls between adjacent floor levels. It was assumed that the principal wall resistance was developed by arching of the 8-in. concrete block and that the contribution of the 4-in. brick veneer to the resistance was negligible because of the 1-in. cavity. Since the window area on each story was small compared with the large room volume, the pressure build-up within the room could not occur in sufficient time to influence the exterior wall response, and therefore the net loading on the wall was assumed to be equal to the exterior blast loading.

Using the information from the on-site survey, it was found necessary to analyze only the following case to estimate the collapse overpressure of the Southern Furniture Exhibition Building:

IF1. Sides A, B, and D, walls on story levels 2 through 10. One-way arching wall.

Interior partitions were not included in the analysis, since, as mentioned previously, there were very few permanent-type partitions on the stories of interest. The dimensions and wall properties used in the analysis are given in Table 2.

The results of the analysis of the Exhibition Building, using the field survey data are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IF1	1.6	0.6	0.9	2.4

Table 2

SOUTHERN FURNITURE EXHIBITION BUILDING
WALL PROPERTY DATA

Case	Location Side	Location Story	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	S (ft) Mean	S (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location of Openings	Delay (msec)	Room Volume (cu ft)
Field survey data														
IF1	ABD	2-10	A One-way	Brick(4) Concrete block(8)	8	132	-	67	-	-	-	-	-	-
Building plan data														
IP1	ABD	2-7, 9,10	A One-way	Brick(4) Concrete block(4)	4	128	-	57	-	-	-	-	-	-
IP2	ABD	8	A One-way	Brick(4) Concrete block(4)	4	212	-	43	-	-	-	-	-	-

* See Table 9 for a key to wall types

Building Plan Data. An examination of the building plans showed that the type of exterior walls in the Exhibition Building was as indicated in the survey data, although there were two differences between the details of the wall design and the survey information. First, the concrete block backing wythe was found to be only 4-in. thick rather than 8-in. thick, which would have a significant effect on wall resistance. Second, the cavity between the brick and concrete block wythes was 2 in. rather than 1 in.

The specific wall analyzed for this phase was the same as that discussed under the survey data. However, even though it was noted in the survey information that the eighth story had a height of 19 ft, it was not analyzed as a separate case. For the analysis using the plan data, it was decided that the increased wall span warranted an additional analysis, and therefore the following two cases were analyzed to estimate the collapse overpressure of the Southern Furniture Exhibition Building:

IP1. Sides A, B, and D, walls on story levels 2 through 7, and 9 and 10. One-way arching wall.

IP2. Sides A, B, and D, walls on story 8. One-way arching wall

The dimensions and wall properties used in the analysis are given in Table 2.

The results of the analysis of the Exhibition Building, using the building plan data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IP1	0.3	0.1	0.2	0.5
IP2	Negligible			

As can be seen in the tabulation, the analysis using the survey data resulted in a mean predicted collapse overpressure for the exterior walls on a typical 12-ft high story of the Exhibition Building that was over five times as high as the predicted value for the same wall using the plan data. The primary reason for this difference in the collapse predictions is the difference in thickness of the concrete block backing wythe, which was given as 8 in. in the survey information but was only 4 in. on the plans. It should be noted that although the relative difference between the two collapse values is large the actual difference of about 1 psi for the case cited may not be too important for the purposes of OCD.

The collapse overpressure for the wall on the eighth story was found to be negligible as a result of the 19-ft story height. Since the wall resistance was of such a low value for the arching mode, the wall was reanalyzed using the bending resistance of the 4-in. thick brick veneer. The collapse strength was also found to be negligible for this case. Because of the low resistance of the wall, there is some question as to whether the wall was actually constructed with the 4-in. thick concrete block backing wythe shown on the drawings.

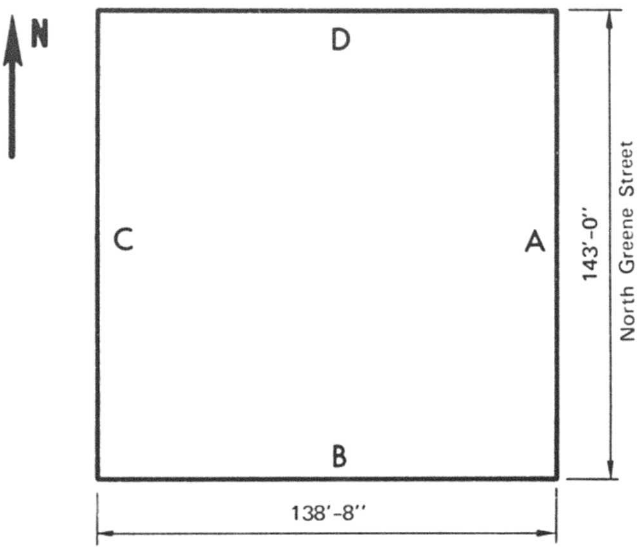
Greensboro Public Library

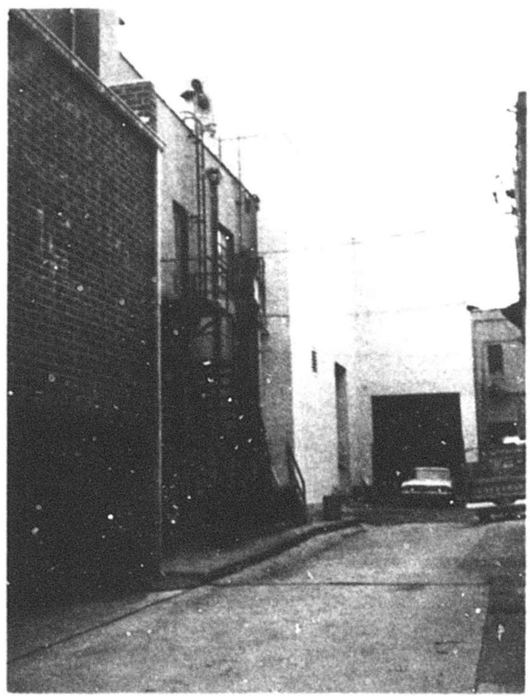
Description

The Greensboro Public Library constructed in 1964, is located at N. Greene and W. Gaston Streets in Greensboro, North Carolina. The Library consists of two stories above ground and two basement levels. The overall height of the building is 33 ft and plan dimensions of 140 ft by 143 ft provide an area of about 17,000 sq ft on the first floor and 20,000 sq ft on the second and basement floor levels. As noted on Figure 2, sides A and B have a minimum window area except for the front entrance. Most of the wall area on sides C and D is shielded by adjacent buildings.

SIDE A

SIDE B





SOURCE: RTI.

SIDES C AND D

FIGURE 2 PHOTOGRAPHS AND PLOT PLAN OF GREENSBORO PUBLIC LIBRARY

The building was constructed with a conventional reinforced concrete frame with beams and columns. On the first and second stories the one-way concrete joist floor system is supported by the frame beams, whereas the floor over the lower basement level is a 6-in. thick solid concrete slab with slab bands. The first floor concrete slab is 3 in. thick.

The exterior walls on sides A and B are constructed with an outer veneer consisting of 4- or 6-in. thick precast stone panels and an inner wythe of 8-in thick solid brick with a 1-in. cavity between. The brick backing is inset in the concrete frame and the stone panels are continuous over the frame. The walls on sides C and D are constructed with a 4-in. brick veneer facing, which is backed with an 8-in. concrete block. The concrete block is inset in the frame and the brick is continuous over the frame members; there is no cavity. The interior partition of primary interest is the 8-in. thick concrete block wall that surrounds the auditorium on the second story. There are also movable type partitions that form office space on the second story, but these are of minor interest for damage and casualty calculations.

Analysis

Field Survey Data. During the on-site survey the exterior walls of the Library were classified as nonreinforced concrete block panel walls with stone veneer on sides A and B and brick veneer on sides C and D. The stone facing was described as mosaic cast stone panels. The 8-in. concrete block was estimated to be inset in the frame and the 4-in. brick or stone panels as continuous over the frame, with no cavity.

The walls on sides A and B were analyzed as unreinforced masonry unit walls with two-way arching between frame members. Since no cavity was specified in the survey data, the concrete block was assumed to be well bonded to the brick or stone veneer, and the total 12-in.-wall thickness was assumed effective in developing the wall resistance.

A separate analysis was made for the walls on the first and second stories because the large auditorium on the second story could modify the effect of the interior room-filling pressure on the exterior wall collapse. Since an adjacent structure shielded part of the wall on side C and all the wall on side D, only the second story wall on side C was analyzed for blast loading. The interior partitions on the second story surrounding the auditorium were analyzed as two-way arching walls since the data indicated they were constructed along the column lines.

Using the information from the on-site survey, it was found necessary to analyze the following four cases to estimate the collapse overpressure of the Greensboro Public Library:

IIF1. Sides A and B, walls on first story. Two-way arching wall.

IIF2. Sides A and B, walls on second story. Two-way arching wall.

IIF3. Side C, walls on second story. Two-way arching wall.

IIF2′. Sides A and B, second-story wall surrounding auditorium. Interior two-way arching wall.

The dimensions and wall properties used in the analysis are given in Table 3.

The results of the analysis of the Library, using the field survey data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IIF1	6.0	1.5	4.1	7.9
IIF2	6.9	1.2	5.4	8.5
IIF3	5.2	1.1	3.8	6.7
IIF2′	2.7	1.5	0.8	4.6

Table 3

GREENSBORO PUBLIC LIBRARY
WALL PROPERTY DATA

Case	Location Side	Location Story	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	S (ft) Mean	S (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location of Openings	Delay (msec)	Room Volume (cu ft)
Field survey data														
IIF1	AB	1	A Two-way	Stone(4) Concrete block(8)	12	150	282	25.0	-	1	795	Front	3	257,400
IIF2	AB	2	A Two-way	Stone(4) Concrete block(8)	12	150	282	15.0	-	1	555	Front	3	181,350
IIF3	C	2	A Two-way	Brick(4) Concrete block(8)	12	150	282	15.0	-	-	-	-	-	-
IIF2´	AB	2	A Two-way	Concrete block(8)	8	16	282	15.0	-	1	555	Front	43	181,350
Building plan data														
IIP1	AB	1	A Two-way	Stone(6) Brick(8)	8	152	284	12.5	-	1	421	Front	3	257,400
IIP2	A	2	A Two-way	Stone(4) Brick(8)	8	152	286	15.0	-	1	273	Front	23	107,700
IIP3	C	2	A Two-way	Brick(4) Concrete block(8)	12	152	286	15.0	-	-	-	-	-	-
IIP4	B	2	A Two-way	Stone(4) Brick(8)	8	152	286	15.0	-	1	102	Front	3	38,700
IIP2´	B	2	A Two-way	Concrete block(8)	8	166	280	15.0	-	1	102	Front	40	38,700

* See Table 9 for a key to wall types.

Building Plan Data. An examination of the building plans indicated several differences between the design of the Library and the data obtained in the field survey. The exterior walls on sides A and B were constructed with either 4- or 6-in. thick precast stone panels with an inner 8-in.-thick brick wythe and a 1-in. cavity. This differed from the survey data where it was noted that the inner wythe was concrete block and that there was no cavity. Because of the cavity between the stone veneer and the inset brick panels, it was assumed for the plan data analysis that the bending strength of the stone panels was negligible compared with the arching resistance of the brick. For sides C and D it was found that the walls were constructed as indicated in the survey data, i.e., a 4-in. thick brick veneer backed with an 8-in. thick concrete block and no cavity.

For the survey data analysis of the interior wall surrounding the auditorium on the second story, it was assumed that the size of the opening through which the blast wave could enter the building was equal to the story height times the length of the diagonal across the corner windows shown in Figure 2.* The plans, however, indicated that the opening into the second story was much less than that assumed because of the existence of an 8-in. thick concrete block wall on the inside of the building that enclosed the main entranceway and circular stairs. Since the wall also extended from the stairs to the auditorium wall, the room volume used for the plan data analysis was only about 20 percent of that used previously, as noted in Table 3.

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* As noted on the sketches furnished with the field survey data in the Appendix, the corner windows enclose the main Library entranceway and circular stairs leading to the second story.

The above difference in room volume also affected the room-filling pressure used in the calculation of the net load on the exterior walls. Therefore, to describe adequately the collapse of the Library, it was necessary to perform a separate analysis for the exterior walls on sides A and B of the second story. Except for this change the specific walls analyzed to estimate the collapse overpressure of the Library for the plan data analysis were the same as those discussed under the survey data analysis and were as follows:

IIP1. Sides A and B, walls on first story. Two-way arching wall.

IIP2. Side A, walls on second story. Two-way arching wall.

IIP3. Side C, walls on second story. Two-way arching wall.

IIP4. Side B, walls on second story. Two-way arching wall.

IIP2′. Sides A and B, second story wall surrounding auditorium. Interior two-way arching wall.

The location of the interior wall surrounding the auditorium is shown in Figure 3. The dimensions and wall properties used in the analysis are given on Table 3.

The results of the analysis of the Library, using the building plan data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IIP1	5.2	1.8	2.8	7.5
IIP2	5.1	1.5	3.2	7.0
IIP3	5.1	1.3	3.4	6.8
IIP4	5.6	1.5	3.7	7.4
IIP2′	3.5	2.3	0.6	6.4

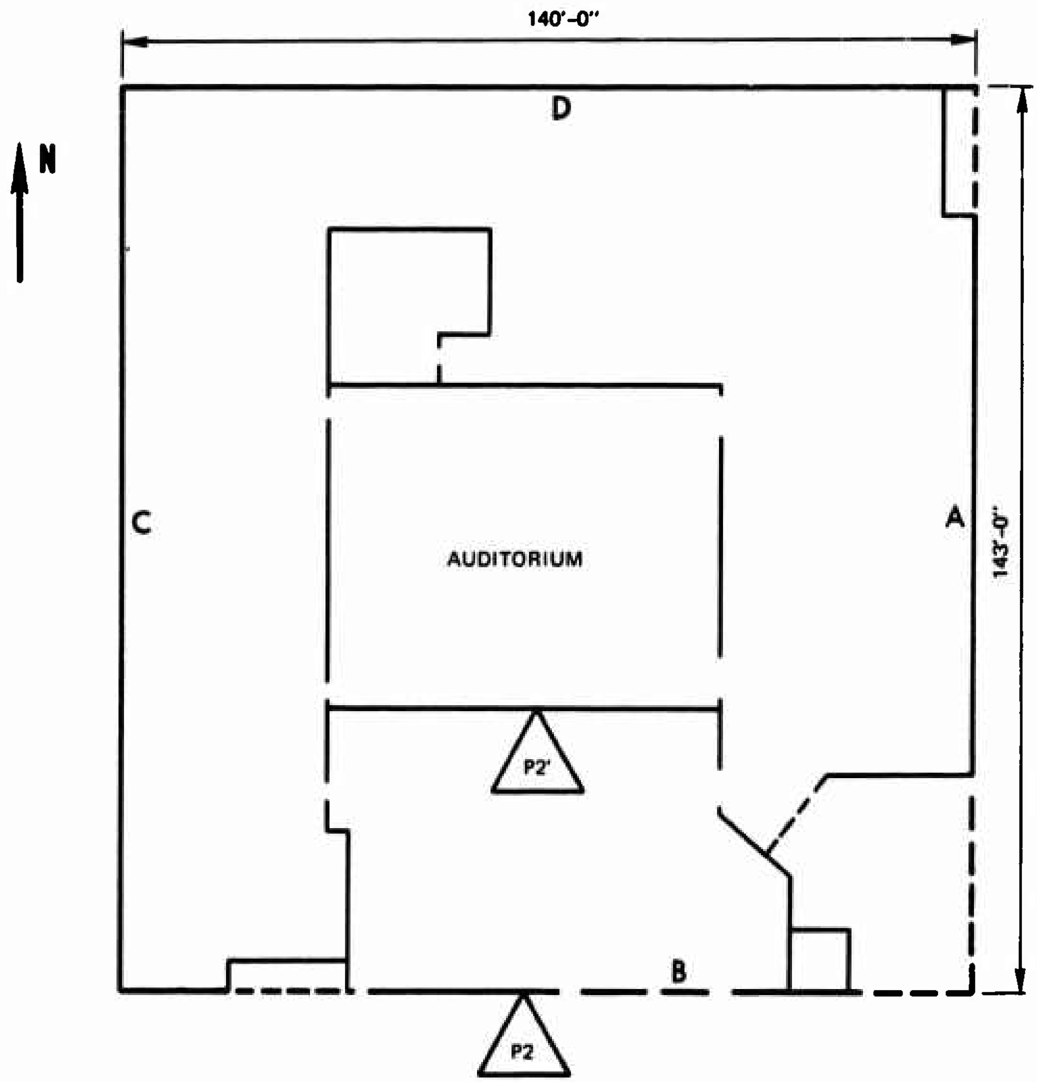


FIGURE 3 PLAN VIEW OF INTERIOR WALLS ON SECOND STORY GREENSBORO PUBLIC LIBRARY

As can be seen in the tabulation, the analysis using the survey data resulted in a prediction of a 50 percent probability of collapse for the exterior walls of the Library that ranged from 2 to 35 percent greater than the predictions made for the same walls using the plan data. For the walls of sides A and B (Cases IIF1 and IIP1) on the first story, the 15 percent increase shown above is misleading because the differences in the wall construction noted previously tend to be compensating. That is, the plans showed that the backing wythe for the stone veneer was brick, which would provide a wall with a greater resistance than the concrete block used in the survey data analysis. Also, since the plans showed that the wall had a 1-in. cavity, the effective wall thickness was 8-in. rather than the 12-in. assumed previously; this, of course, would tend to decrease the collapse prediction for the plan data analysis.

In addition to the above two factors, the collapse prediction for the second story walls on side A (Cases IIF2 and IIP2) was affected by the room volume used in the analyses, as noted in Table 3. This resulted in a 50 percent probability of collapse for the survey data that was 35 percent greater than that for the plan data.

For the interior wall surrounding the auditorium (Cases IIF2′ and IIP2′), the analysis using the building plan data resulted in a prediction for the mean collapse overpressure that was about 30 percent greater than that made with the survey data. Since the wall properties were similar for both cases, as noted in Table 3, it is evident that the difference resulted from the variation in the area of openings and room volume, which would affect the pressure-time history on the walls.

Laura Cone Dormitory

Description

The Laura Cone Dormitory, constructed in 1967, is located on West Market Street, U.N.C.-G, Greensboro, North Carolina. The building consists of 9 stories and a ground or basement floor. The overall height of the building is 98 ft and plan dimensions of 64 ft by 194 ft provide an area of about 5300 sq ft on the ground floor and 8300 sq ft on the upper floors. Figure 4 shows the exterior walls and general window layout for the Dormitory.

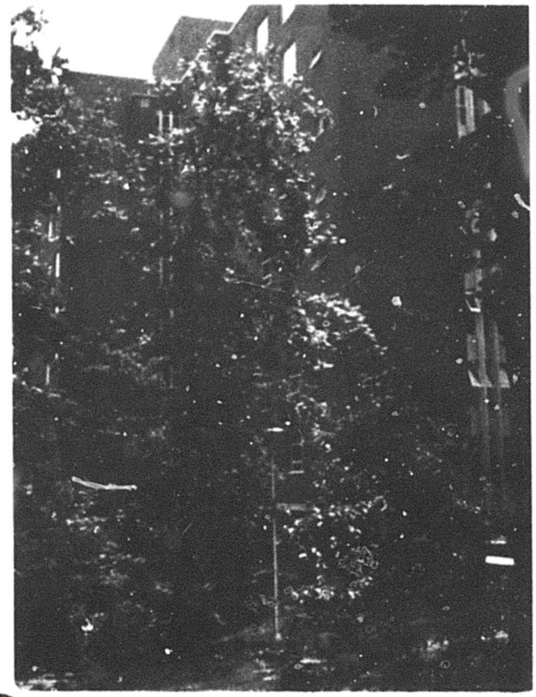
The Dormitory has a structural steel frame with riveted and welded column and beam connections. The floors are 2-3/4-in. thick concrete on galvanized corrugated steel forms that are supported by open-web steel joists spanning between the frame beams.

The exterior walls on sides A and C are constructed with a 4-in. thick brick veneer with a 4-in. thick concrete block backing wythe and a 2-in. cavity. The walls are unreinforced and the 4-in. concrete block is inset between floor beams; the brick veneer is continuous over the floors although supported on shelf angles at each floor level. On sides B and D the 4-in. thick brick veneer is backed with 8-in. thick concrete block and there is no cavity. The concrete block is inset in the frame and the brick is continuous over the frame members. The interior partitions are constructed with unreinforced concrete block, 4-in. thick between rooms and 6-in. thick in the corridors. The partitions are nonload bearing, even though there is some wedging between the top of the room partitions and the floor beams.

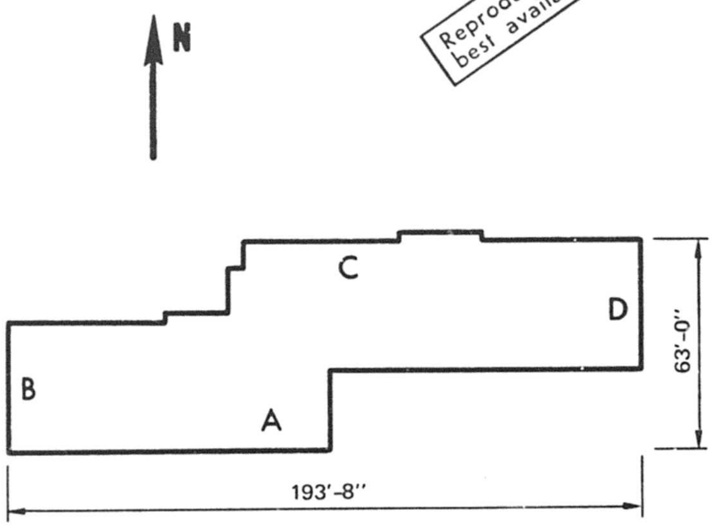
Analysis

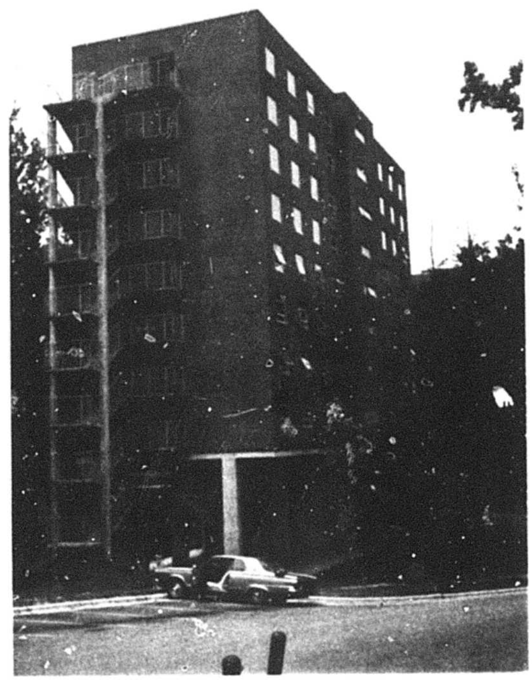
Field Survey Data. During the on-site survey the exterior walls of the Dormitory were classified as nonreinforced concrete block panel walls with brick masonry veneer. The concrete block on the upper floors was

SIDE A



SIDES B AND C





SOURCE: RTI.

SIDES C AND D

FIGURE 4 PHOTOGRAPHS AND PLOT PLAN OF LAURA CONE DORMITORY

described as 6-in. thick and inset in the frame; there was a limited amount of 8-in. concrete block on the basement and first story levels. The brick veneer was 4-in. thick with a 1-in. cavity on all sides of the building.

The exterior walls on sides A and C were analyzed as unreinforced masonry unit walls with two-way arching between frame members. It was assumed that the principal wall resistance was developed by arching of the 6-in. concrete block and that the contribution of the 4-in. brick veneer to the resistance was negligible because of the 1-in. cavity. Since there were no windows on sides B and D except in the corridor area, a separate analysis was made for these walls. However, no separate analysis was made for the limited number of walls with the 8-in. thick concrete block backing wythe on the first story. The interior partitions on the upper floors between the rooms and the corridor were analyzed as one-way arching walls, even though they were described as load-bearing walls in the survey data.

Using the information from the on-site survey, it was found necessary to analyze the following three cases to estimate the collapse overpressure of the Laura Cone Dormitory:

IIIF1. Sides A and C, walls on story levels 2 through 9. Two-way arching wall.

IIIF2. Sides B and D, walls on story levels 2 through 9. Two-way arching wall.

IIIF1′. Sides A and C, walls on story levels 2 through 9. Interior one-way arching wall.

The dimensions and wall properties used in the analysis are given in Table 4.

Table 4

LAURA CONE DORMITORY
WALL PROPERTY DATA

Case	Location		Wall	Material	t_w	L_V	L_H	S (ft)		Number of	Area of Openings	Location of	Delay	Room Volume
	Side	Story	Type*		(in.)	(in.)	(in.)	Mean	Standard Deviation	Openings	(sq ft)	Openings	(msec)	(cu ft)
Field survey data														
IIIF1	AC	2-9	A Two-way	Brick(4) Concrete block(6)	6	96	138	10.7	5.0	1	34	Front	3	1,470
IIIF2	BD	2-9	A Two-way	Brick(4) Concrete block(6)	6	96	192	16.0	-	1	34	Side	8	1,470
IIIF1′	AC	2-9	A One-way	Concrete block(6)	6	108	-	10.7	5.0	1	34	Front	3	1,470
Building plan data														
IIIP1	AC	2-9	U-3	Brick(4) Concrete block(4)	4	98	144	10.7	5.0	1	36	Front	3	2,320
IIIP2	BD	2-9	A One-way	Brick(4) Concrete block(8)	12	96	-	16.7	-	1	36	Side	8	1,700
IIIP1′	AC	2-9	U-1	Concrete block(6)	6	96	48	10.7	5.0	1	36	Front	3	2,320

* See Table 9 for a key to wall types.

The results of the analysis of the Laura Cone Dormitory, using the field survey data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IIIF1	7.6	2.0	5.0	10.1
IIIF2	5.4	0.8	4.3	6.4
IIIF1′	2.8	0.8	1.7	3.8

Building Plan Data. An examination of the building plans indicated several differences between the design of the Dormitory and the data obtained in the field survey. The exterior walls on Sides A and C were constructed with a 4-in. thick brick veneer and a 4-in. thick concrete block backing wythe and a 2-in. cavity; this differed from the survey data where it had been found that the inner wythe was 6-in. thick concrete block. The brick veneer is supported on shelf angles at each floor level, and the concrete block is inset between floor levels* although the wall is not in direct contact with the spandrel beams at the top of the wall. Therefore, for the analysis it was assumed that the resistance of the wall was controlled by the bending strength of the brick veneer and that the contribution of the concrete block to the wall resistance was negligible. It was also found that the exterior walls on sides B and D were not constructed as noted in the survey data but consisted of a 4-in. thick brick veneer backed with an 8-in. thick concrete block and no cavity. For the analysis it was assumed that only one-way arching could develop between spandrel beams and that the total 12-in. thickness of the wall would contribute to the arching resistance.

* The column lines on sides A and C were about 7 ft behind the plane of the exterior walls, and the floor beams were cantilevered from the columns.

Since the plans indicated that the corridor walls were not in direct contact with the floor beams, it was assumed in the analysis that the wall resistance was developed through bending rather than arching, as assumed for the survey data analysis. Furthermore, since the corridor walls are inadequately supported to develop the wall resistance for a blast wave approaching from the direction of the window, the volume used in the collapse predictions for both the interior and exterior walls was equal to the room volume plus the volume of the adjacent corridor.

The specific walls analyzed to estimate the collapse overpressure of the Dormitory for the plan data analysis were the same as those discussed under the survey data analysis and were as follows:

IIIP1. Sides A and C, walls on story levels 2 through 9. Two-way unreinforced masonry unit wall fixed on vertical edges and simply supported on horizontal edges without arching.

IIIP2. Sides B and D, walls on story levels 2 through 9. One-way arching wall.

IIIP1′. Sides A and C, walls on story levels 2 through 9. Interior two-way unreinforced masonry unit wall with simple supports and without arching.

The interior wall analyzed for a blast wave striking side A is indicated on Figure 5. The dimensions and wall properties used in the analysis are given in Table 4.

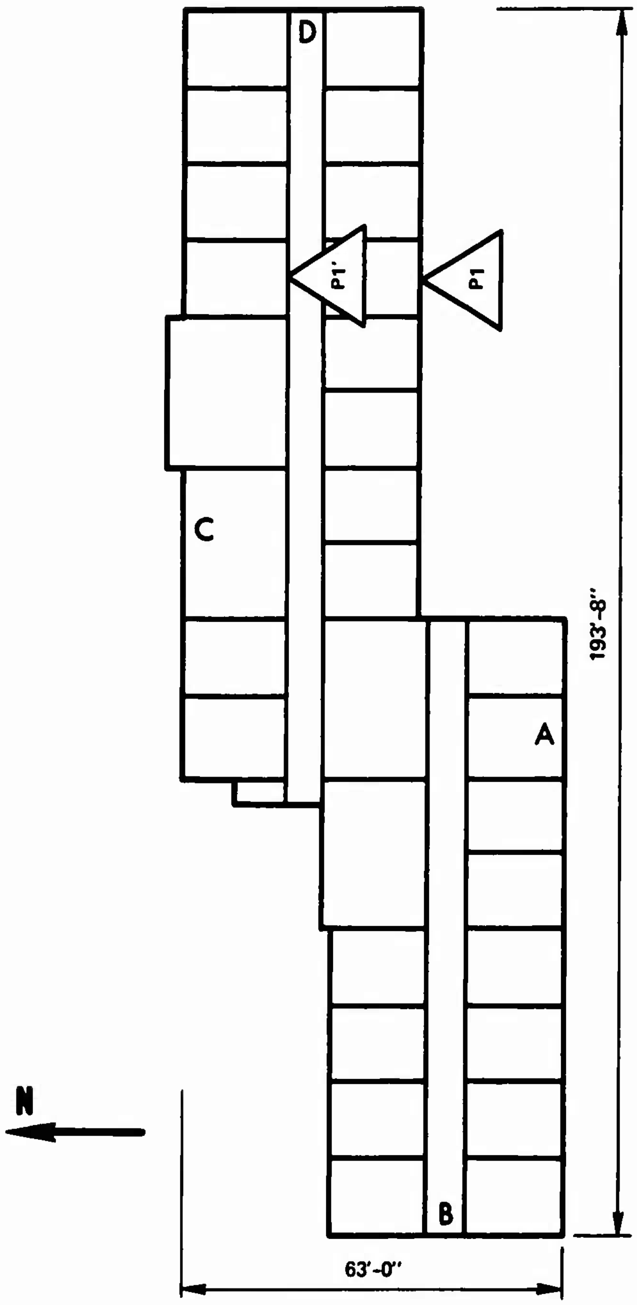


FIGURE 5 PLAN VIEW OF INTERIOR WALLS ON UPPER STORIES
LAURA CONE DORMITORY

The results of the analysis of the Dormitory, using the building plan data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IIIP1	1.0	0.1	0.9	1.1
IIIP2	11.2	1.1	9.7	12.6
IIIP1′	0.7	0.3	0.3	1.0

As can be seen in the tabulations for the survey and plan data analyses, the difference in predicted collapse overpressures ranged from a factor of about one-half to eight. For the exterior walls on sides A and C (Cases IIIF1 and IIIP1), the analysis with the survey data resulted in a predicted collapse overpressure that was almost eight times that made with the plan data. This large difference resulted from the differences in wall construction, thickness, and support conditions discussed previously in this subsection. For the exterior walls on sides B and D (Cases IIIF2 and IIIP2), the analysis with the survey data resulted in a collapse overpresure that was about one-half that with the plan data. This was due primarily to the difference in wall thickness used in the two analyses, and the discrepancy would have been greater if the support conditions had been identical, i.e., if Case IIIF2 was two-way arching and/or if Case IIIP2 was one-way arching.

The predicted collapse overpressure for the interior corridor walls using the survey data can be seen to be four times that obtained using the plan data. This resulted solely from the different support conditions used. For the survey data analysis it was assumed that the interior partitions could arch between floor beams, but the plans showed that the top of the corridor wall was not in direct contact with the floor beams and the wall would therefore develop its resistance through bending.

Willa B. Player Hall

Description

The Willa B. Player Hall, constructed in 1966, is a student dormitory located at Bennett College, Greensboro, North Carolina. The building consists of 2 stories and a lower level below the main floor level. The overall height of the building is about 35 ft to the eave line, and plan dimensions of 65 ft by 205 ft provide an area of about 11,800 sq ft on each floor level. Figure 6 shows a location plan for the Hall, and Figure 7 shows the window and wall area on the four sides of the building. Note that the lower level is not fully exposed on all sides.

The building has a load-bearing exterior wall and interior structural steel columns and beams. The floors are 2-1/2-in. thick concrete on standard corrugated steel forms that are supported by open-web steel joists spanning between the exterior wall and the interior beams.

The exterior walls are load-bearing unreinforced masonry unit walls and are of similar construction on all sides of the building. On the lower level the walls are 16-in. thick solid brick and on the upper two stories the walls consist of a 4-in. thick brick facing with an 8-in. thick concrete block backing wythe; the brick and block are fully bonded. The interior partitions in the corridors and between the rooms are constructed with 8-in. thick unreinforced concrete block. The corridor walls were inset between the frame column and beams, and the room partitions were nonload bearing and were supported by a double floor joist.

Analysis

Field Survey Data. During the on-site survey the exterior walls of the Hall were classified as nonreinforced brick bearing walls without masonry veneer. The wall on the lower level was described as 16-in. thick solid brick and that on the other two levels as 12-in. thick solid brick.

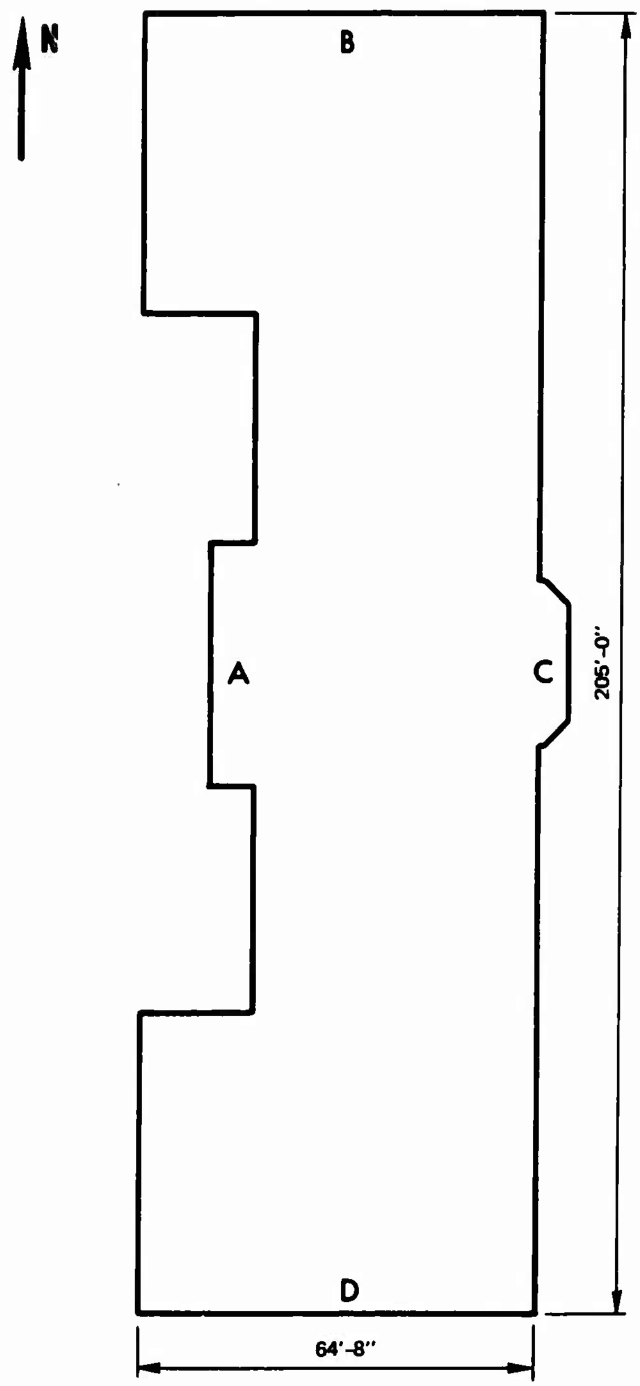
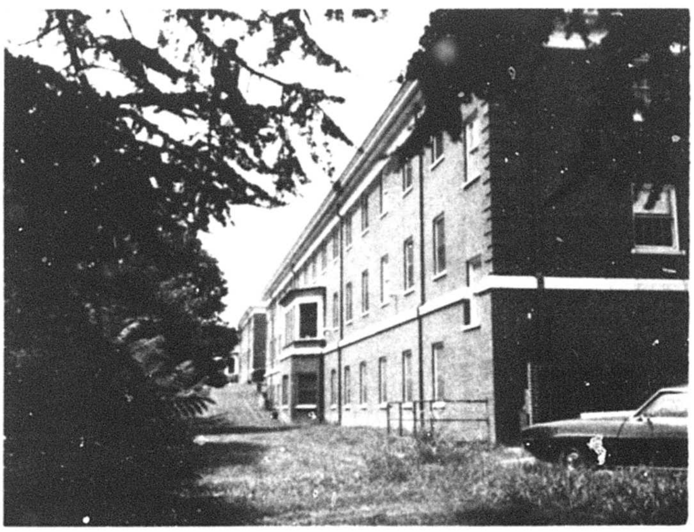


FIGURE 6 PLOT PLAN OF WILLA B. PLAYER HALL

SIDE A

SIDES A AND B



SIDES B AND C

SIDE D

SOURCE: RTI.

FIGURE 7 PHOTOGRAPHS OF WILLA B. PLAYER HALL

All exterior walls were analyzed as unreinforced masonry unit walls without arching. Since the walls were of the load-bearing type, it was necessary to analyze the walls on each story to account for the difference in vertical in-plane forces resulting from the building dead load. Since the interior walls were also classified as load-bearing, analyses were made for the interior corridor partitions on each story.

Using the information fiom the on-site survey, it was found necessary to analyze the following nine cases to estimate the collapse overpressure of the Willa B. Player Hall:

IVF1. Side A, wall on left wing of first story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVF2. Sides B and C, wall on first story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVF3. Side A, wall on left and right wings of second story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVF4. Sides B, C, and D,* wall on second story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVF5 Side A, wall on left and right wings of third story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVF6. Sides B, C, and D,* wall on third story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

---

* Wall in center portion of side A is similar to this case.

IVF2′. All interior partitions on corridors of first story. Two-way unreinforced masonry unit wall with simple supports and without arching.

IVF4′. All interior partitions on corridors on second story. Two-way unreinforced masonry unit wall with simple supports and without arching.

IVF6′. All interior partitions on corridors on third story. Two-way unreinforced masonry unit wall with simple supports and without arching.

The interior partitions between adjacent rooms were not analyzed separately from the corridor partitions since the survey data indicated that the walls were of similar construction. The dimensions and wall properties used in the field survey data analysis are given in Table 5.

The results of the analysis of the Hall, using the field survey data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IVF1	7.7	0.7	6.9	8.6
IVF2	8.3	0.6	7.5	9.1
IVF3	4.9	0.5	4.2	5.5
IVF4	5.2	0.3	4.8	5.7
IVF5	3.7	0.2	3.4	4.0
IVF6	3.7	0.2	3.4	3.9
IVF2′	1.9	0.1	1.8	2.0
IVF4′	1.3	0.03	1.24	1.32
IVF6′	0.5	0.04	0.43	0.52

Table 5

WILLA B. PLAYER HALL
WALL PROPERTY DATA FROM FIELD SURVEY DATA

Case	Location		Wall	Material	t_w	L_V	L_H	P_v	S (ft)		Number	Area of	Location	Delay	Room
	Side	Story	Type*		(in.)	(in.)	(in.)	(lb/ in.)	Mean	Standard Deviation	of Openings	Openings (sq ft)	of Openings	(msec)	Volume (cu ft)
Field survey data															
IVF1	A	1	U-2	Brick(16)	16	144	252	280	15.0	5.5	2	19/19	Front/side	3/8	3,020
IVF2	BC	1	U-2	Brick(16)	16	144	144	320	13.9	6.1	1	19	Front	3	3,020
IVF3	A	2	U-2	Brick(12)	12	132	252	146	15.0	5.0	2	19/19	Front/side	3/8	3,020
IVF4	BCD	2	U-2	Brick(12)	12	132	144	170	11.9	6.1	1	19	Front	3	3,020
IVF5	A	3	U-2	Brick(12)	12	132	252	12	8.5	2.1	2	19/19	Front/side	3/8	3,020
IVF6	BCD	3	U-2	Brick(12)	12	132	144	20	7.8	2.6	1	19	Front	3	3,020
IVF2′	ABCD	1	U-1	Concrete block(8)	8	144	144	204	13.9	6.1	1	19	Front	3	3,020
IVF4′	ABCD	2	U-1	Concrete block(8)	8	132	144	112	11.9	6.1	1	19	Front	3	3,020
IVF6′	ABCD	3	U-1	Concrete block(8)	8	132	144	20	7.8	2.6	1	19	Front	3	3,020

* See Table 9 for a key to wall types.

Building Plan Data. An examination of the building plans indicated several differences between the design of the Hall and the data obtained in the field survey. The exterior walls on the first floor were constructed of 16-in. thick solid brick as noted in the survey data. However, the plans showed that the walls on the upper two stories were constructed with a 4-in. thick brick veneer and an 8-in. thick concrete block backing wythe rather than with the 12-in. thick solid brick as found in the field survey. The exterior walls were of the load-bearing type, as assumed in the survey data analysis.

Since the plans indicated that the corridor walls were inset between the interior beams and columns of the frame, it was assumed in the plan data analysis that the wall resistance was developed through arching between beams. This differed from the survey data analysis where the corridor walls were assumed to be of the load-bearing type. The plans also showed that the concrete block partitions between rooms were supported by double steel joists at each floor level. Since open-web joists cannot effectively transfer vertical forces between stories, the partitions were analyzed as nonload-bearing, simply supported walls. This is in contrast to the survey data analysis where these walls were assumed to be of the load-bearing type.

The specific walls analyzed to estimate the collapse overpressure of the dormitory Hall for the plan data analysis were similar to those discussed under the survey data analysis, except for minor differences in the interior partitions analyzed, and were as follows:

IVP1. Side A, wall on left wing of first story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVP2. Sides B and C, wall on first story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVP3. Side A, wall on left and right wings of second story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVP4. Sides B, C, and D,* wall on second story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVP5. Side A, wall on left and right wings of third story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVP6. Sides B, C, and D,* wall on third story. Two-way unreinforced masonry unit wall with fixed-edge supports and without arching.

IVP3′. Side A, interior wall between rooms on stories 1 through 3. Two-way unreinforced masonry unit wall, with simple supports and without arching.

IVP4′. All interior partitions on corridors on stories 1 through 3. One-way arching wall.

The location of the interior partitions analyzed with the plan data is shown on Figure 8 for the second story. The dimensions and wall properties used in the plan data analysis are given in Table 6.

The results of the analysis of the Hall, using the building plan data, are:

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* Wall in center portion of side A is similar to this case.

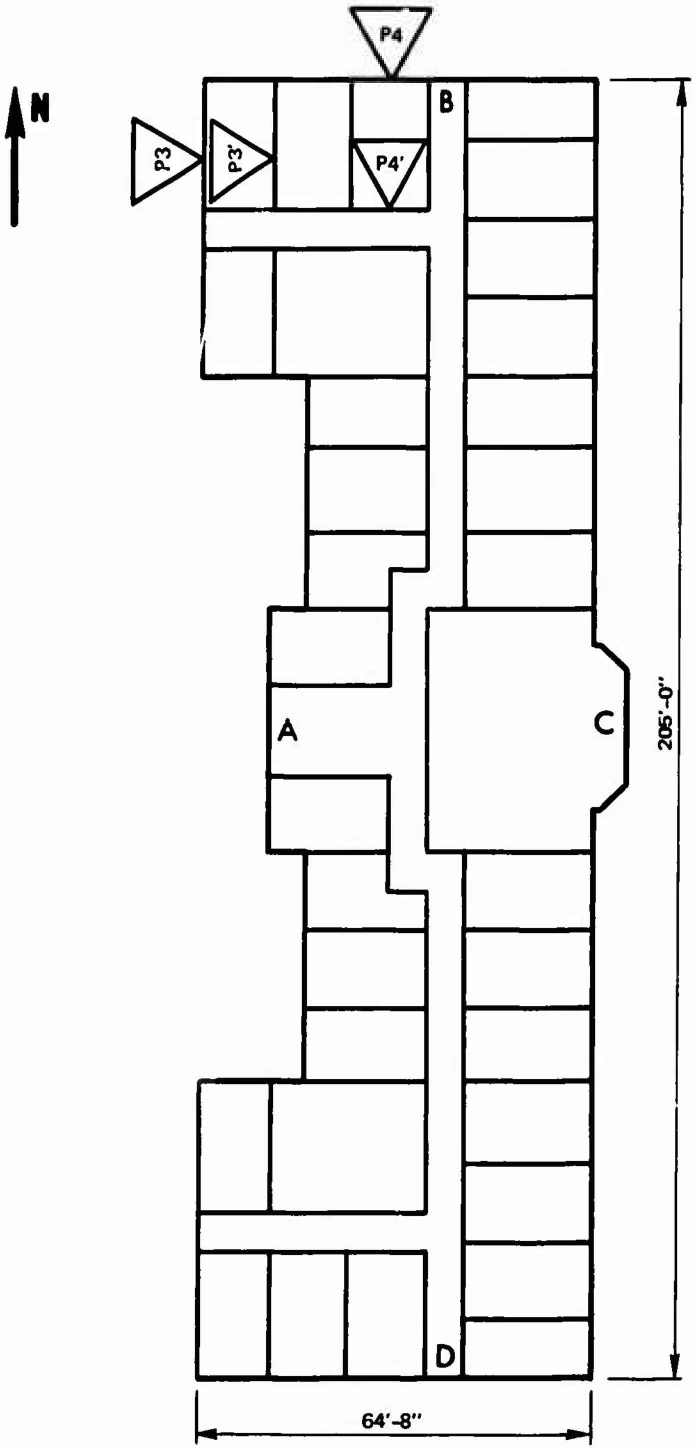


FIGURE 8 PLAN VIEW OF INTERIOR WALLS ON SECOND STORY
WILLA B. PLAYER HALL

Table 6

WILLA B. PLAYER HALL
WALL PROPERTY DATA FROM BUILDING PLAN

Case	Location Side	Location Story	Wall Type*	Material	t_w (in.)	L_V (in.)	L_H (in.)	P_v (lb/in.)	S (ft) Mean	S (ft) Standard Deviation	Number of Openings	Area of Openings (sq ft)	Location of Openings	Delay (msec)	Room Volume (cu ft)
Building plan data															
IVP1	A	1	U-2	Brick(16)	16	143	226	170	13.0	4.3	2	20/20	Front/side	3/8	2,640
IVP2	BC	1	U-2	Brick(16,	16	143	143	270	14.5	6.4	1	20	Front	3	2,640
IVP3	A	2	U-2	Brick(4) Concrete block(8)	12	129	226	90	13.0	4.3	2	20/20	Front/side	3/8	2,420
IVP4	BCD	2	U-2	Brick(4) Concrete block(8)	12	129	140	150	13.0	5.5	1	20	Front	3	2,420
IVP5	A	3	U-2	Brick(4) Concrete block(8)	12	138	226	0	9.0	1.8	2	20/20	Front/side	3/8	2,530
IVP6	BCD	3	U-2	Brick(4) Concrete block(8)	12	138	140	20	9.5	2.5	1	20	Front	3	2,530
IVP3′	A	1-3	U-1	Concrete block(8)	8	118	226	0	13.0	4.3	2	20/20	Front/side	3/8	2,420
IVP4′	ABCD	1-3	A One-way	Concrete block(8)	8	118	-	-	13.0	5.5	1	20	Front	3	2,420

* See Table 9 for a key to wall types.

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
IVP1	7.7	0.7	6.7	8.6
IVP2	8.3	0.7	7.4	9.3
IVP3	4.6	0.5	4.0	5.2
IVP4	5.0	0.5	4.4	5.6
IVP5	3.1	0.2	2.8	3.3
IVP6	3.3	0.1	3.1	3.5
IVP3′	0.2	0.1	0.1	0.3
IVP4′	4.6	2.4	1.5	7.7

As can be seen from the tabulations for the survey and plan data analyses, the predicted collapse overpressure for the exterior walls differs very little for the two sets of data, being a maximum of about 19 percent for the third story of side A (Cases IVF5 and IVP5). Only small differences in the predictions would be expected, of course, since there were only minor differences in the wall properties used in the two analyses, as noted in Tables 5 and 6. It is also apparent that the relatively large differences in the modulus of rupture, f_r, for brick and concrete block (Table 1) had only a minor influence on the collapse strength of load-bearing walls under dynamic load. This results primarily from the fact that the influence of the vertical in-plane forces on the wall resistance is much greater than the influence of the flexural strength of the wall.*

* See Ref. 1, Figure 29, for the results of a sensitivity analysis of the relative effect of the modulus of rupture and vertical in-plane load on the dynamic strength of an unreinforced masonry unit wall.

As mentioned previously for the survey data, the interior partitions along the corridors were analyzed for each story level but a separate analysis was not performed for the partitions between adjacent rooms because both walls had been described as of similar construction. For the interior corridor partitions, the predicted collapse overpressure for the plan data analysis was 4.6 psi for all three stories (Case IVP4′); this value ranges from 2.4 to 9.2 times those obtained with the survey data (Cases IVF2′, 4′, and 6′). The primary reason for this difference results from the support conditions assumed for the two analyses. That is, for the survey data analysis it was assumed that the corridor walls were of the load-bearing wall type--this accounts for the variation of values with story height--but for the plan data the walls were assumed to arch between floor beams.

For the interior partitions between rooms, the predicted collapse overpressure for the plan data analysis was only 0.2 psi (Case IVP3′), which is 0.1 to 0.4 of the values obtained with the survey information (Cases IVF2′, 4′, and 6′). This difference was also a result of the difference in assumed support conditions for the two analyses; i.e., the plans showed that the partition between rooms developed their resistance in bending only, without the effect of the vertical in-plane forces that were included for the load-bearing wall in the survey data case.

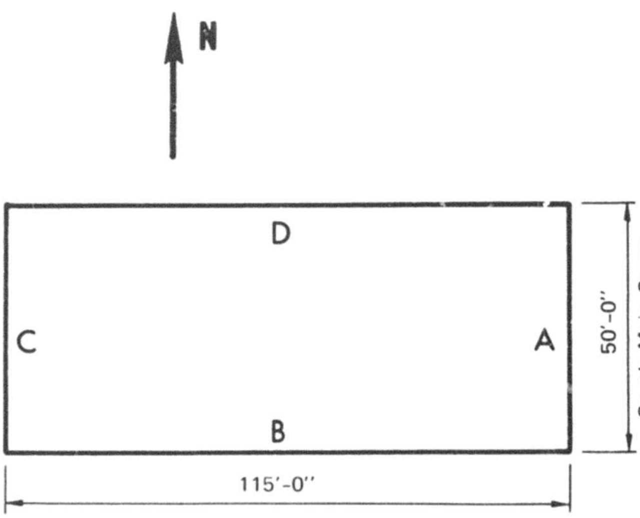
North Carolina National Bank

Description

The North Carolina National Bank, constructed in 1922, is located on South Main Street, High Point, North Carolina. The building consists of 8 stories and an unexposed basement; there is a mezzanine between the first and second stories. The overall height of the building is about 110 ft and plan dimensions of 50 ft by 115 ft provide an area of 5750 sq ft on each floor level. Figure 9 shows the exterior walls and

SIDES A AND B

SIDES B AND C



SIDE D

SOURCE: RTI.

FIGURE 9 PHOTOGRAPHS AND PLOT PLAN OF NORTH CAROLINA NATIONAL BANK

general window layout of the bank. Note that many of the windows on sides B and C have been bricked in on the first story.

The Bank has a structural steel frame with riveted and bolted column and beam connections. The ribbed floor system has a 4-in. thick concrete slab and 4- or 6-in. thick clay tile fillers.

The exterior walls on sides A and B of the first story are 17-in. thick and are constructed with a granite veneer and a brick backing. On sides C and D of the first story the walls are generally 17-in. thick solid brick. On the upper stories the walls are constructed with a 4-in. thick brick veneer and an 8-in. thick terra cotta backing. As can be noted in Figure 9, the exterior column lines on the upper stories of sides A and B are faced with a granite veneer. For all exterior walls the facing is continuous over the frame members and the backing is inset in the frame. The interior partitions on the first story and mezzanine are constructed with unreinforced terra cotta, either 3- or 6-in. thick. On the upper stories the interior partitions are mostly 3-in. unreinforced terra cotta. The partitions are nonload bearing and have numerous openings that have been filled-in with light wood paneling.

Analysis

Field Survey Data. During the on-site survey the exterior walls of the Bank were classified as tile panel walls with stone veneer; all walls were described as 12-in. thick. The 8-in. thick tile backing was estimated to be inset in the frame and the 4-in. thick stone veneer was continuous over the frame; there was no cavity.

The exterior walls on all sides were analyzed as unreinforced masonry unit walls with either one- or two-way arching. For Side A of the first story it was assumed that, because of the many openings, only one-way arching could develop between floor beams on the first and mezzanine stories. On side B it was assumed that one-way arching would develop in

the walls between windows. Furthermore, it was assumed that the bricked-in windows would not contribute to the arching strength of the walls but would remain in place for a sufficient length of time to influence the blast loading and room filling.

The interior partitions on the upper floors consisted primarily of 3-in. thick gypsum block except for the area around the stairs and elevators. Since the partitions were nonload bearing and contained numerous openings, they were considered of insufficient strength to be a hazard and were therefore not analyzed. Also, for the calculation of the interior loading for the analysis of the exterior walls, it was assumed that the interior partitions collapsed rapidly and did not influence the loading significantly.

Using the information from the on-site survey, it was found necessary to analyze the following three cases to estimate the collapse overpressure of the North Carolina National Bank:

VF1. Side A, wall on first story. One-way arching wall.

VF2. Side B, wall on first story. One-way arching wall.

VF3. All sides, walls on upper stories. Two-way arching wall.

The dimensions and wall properties used in the analysis are given in Table 7.

The results of the analysis of the Bank, using the field survey data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
VF1	3.9	0.7	3.0	4.8
VF2	1.8	0.2	1.6	2.0
VF3	12.4	2.6	9.0	15.7

Table 7

NORTH CAROLINA NATIONAL BANK
WALL PROPERTY DATA

Case	Location		Wall	Material	t_w	L_V	L_H	S (ft)		Number of Openings	Area of Openings (sq ft)	Location of Openings	Delay (msec)	Room Volume (cu ft)
	Side	Story	Type*		(in.)	(in.)	(in.)	Mean	Standard Deviation					
Field survey data														
VF1	A	1	A One-way	Stone(4) Tile(8)	13	180	-	11.9	6.1	2	480/360	Front/side	3/12	172,500
VF2	B	1	A One-way	Stone(4) Tile(8)	13	360	-	30.0	-	2	720/120	Front/side	3/11	172,500
VF3	ABCD	2-8	A Two-way	Brick(4) Tile(8)	13	114	168	11.1	5.4	2	40/40	Front/rear	3/53	7,500
Building plan data														
VP1	A	1	A One-way	Stone Brick	17	180	-	12.5	5.2	2	240/225	Front/side	3/10	172,500
VP2	B	1	A One-way	Stone Brick	17	324	-	30.5	-	2	450/240	Front/side	3/20	172,500
VP3	ABCD	2-8	A Two-way	Brick(4) Tile(8)	13	102	180	11.1	5.4	1	48	Front	3	7,570

* See Table 9 for key to wall types.

Building Plan Data. An examination of the building plans indicated several differences between the design of the Bank and the data obtained in the field survey. The exterior walls on sides A and B of the first story were constructed with a granite veneer and a brick backing and were 17-in. thick; this differed from the survey data where it had been found that the backing wythe was clay tile and that the wall thickness was only 13 in. Since the plan data showed that the brick was inset in the frame, as noted in the survey data for the clay tile, the first-story walls were analyzed as one-way arching walls. Although not shown in detail on the plans, the granite was evidently well bonded to the brick.

The plans indicated that all exterior walls on the upper stories were constructed as noted in the survey, i.e., with an 8-in. thick clay tile backing inset in the structural frame and a 4-in. thick brick veneer continuous over the frame members. As mentioned in the survey data analysis, the interior partitions were primarily constructed of 3-in. thick clay tile and were therefore not considered as a structural member for building collapse predictions.

The specific walls analyzed to estimate the collapse overpressure of the Bank for the plan data analysis were the same as those discussed under the survey data analysis and were as follows:

VP1. Side A, wall on first story. One-way arching wall.

VP2. Side B, wall on first story. One-way arching wall.

VP3. All sides, walls on upper stories. Two-way arching wall.

The dimensions and wall properties used in the analysis are given in Table 7.

The results of the analysis of the Bank, using the building plan data, are:

	Predicted Collapse Overpressure, psi			
Case	Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
VP1	16.4	4.2	11.0	21.8
VP2	5.4	0.7	4.6	6.3
VP3	15.7	4.0	10.5	20.8

As can be seen from the tabulations, the mean predicted collapse overpressures for the exterior walls ranged from about 27 to 320 percent greater for the plan data analysis than for the survey data analysis. The largest difference in predicted values occurred for the exterior walls of the first story of side A (Cases VF1 and VP1). As noted from the wall property data in Table 7, this difference in collapse values can be attributed to the 4-in. thicker wall used in the plan data analysis and to the difference in wall construction; i.e., the plans showed that the backing wythe was solid brick rather than structural clay tile as indicated in the survey information. These same factors also account for the difference in the predicted collapse overpressures ior the exterior walls on the first story of side B (Cases VF2 and VP2).

The relatively small difference in the predicted collapse overpressure for the upper story exterior walls (Cases VF3 and VP3) was as would be expected because there were only minor differences in the wall properties used in the two analyses, as noted in Table 7.

As mentioned in Section I, to be able to use the exterior wall models for predicting building collapse, it was necessary to assume for the analysis that the structural frame did not collapse. Since the incident overpressure required to collapse the exterior wall on the upper stories of the Bank is almost 16 psi for the mean value, the structure will be

subjected to large lateral forces during both the diffraction and drag phases, for which it was not designed. Since the overall height of the building is 110 ft and since it is only 50 ft wide in the short direction, it is possible that the frame may experience a failure at a lower overpressure than that predicted for the collapse of the exterior walls.

III SUMMARY AND DISCUSSION

The predicted collapse overpressure for all five Greensboro buildings and for both the field survey and building plan data analyses are summarized in Table 8; Table 9 gives the key to the wall types and support case designations. A comparison of the results of the analyses demonstrates that when the proper building information is obtained in an on-site field survey, there is then generally good agreement between the collapse predictions made with both the field survey and building plan data. On the other hand, if certain construction details are not documented correctly, especially the wall support conditions and thickness, then the predictions from the two sets of data can vary by a wide degree.

A good example of the influence of the wall support condition on the collapse overpressure can be shown by the results of the analysis of Willa B. Player Hall. The building is of the load-bearing wall type and the exterior walls were described in the survey information as solid brick throughout, with a 16-in. thickness on the first story and a 12-in. thickness on the second and third stories. The wall support conditions and thickness were in agreement for the two sets of data, but the plans showed that the exterior walls on the second and third stories were constructed with a 4-in. thick brick veneer and an 8-in. thick concrete block backing wythe rather than solid brick. Even with this difference in construction, the maximum difference in the predicted collapse overpressure for the survey and plan data analysis was only about 19 percent for the exterior wall cases IVF5 and IVP5.*

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* As can be seen for the exterior wall cases in Tables 5 and 6, there are other minor differences in the wall properties that were obtained from the survey and plan data.

Table 8

SUMMARY OF WALL ANALYSES

Case*	Location: Side	Location: Story	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
Southern Furniture Exhibition Building								
IF1	ABD	2-10	A One-way	13	1.6	0.6	0.9	2.4
IP1	ABD	2-7, 9,10	A One-way	10	0.3	0.1	0.2	0.5
IP2	ABD	8	A One-way	10	Neglibible			
Greensboro Public Library								
IIF1	AB	1	A Two-way	12	6.0	1.5	4.1	7.9
IIF2	AB	2	A Two-way	12	6.9	1.2	5.4	8.5
IIF3	C	2	A Two-way	12	5.2	1.1	3.8	6.7
IIF2′	AB	2	A Two-way	8	2.7	1.5	0.8	4.6
IIP1	AB	1	A Two-way	8	5.2	1.8	2.8	7.5
IIP2	A	2	A Two-way	8	5.1	1.5	3.2	7.0
IIP3	C	2	A Two-way	12	5.1	1.3	3.4	6.8
IIP4	B	2	A Two-way	8	5.6	1.5	3.7	7.4
IIP2′	A	2	A Two-way	8	3.5	2.3	0.6	6.4

Table 8 (continued)

Case*	Location Side	Location Story	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
Laura Cone Dormitory								
IIIF1	AC	2-9	A Two-way	6	7.6	2.0	5.0	10.1
IIIF2	BD	2-9	A Two-way	6	5.4	0.8	4.3	6.4
IIIF1′	AC	2-9	A One-way	6	2.8	0.8	1.7	3.8
IIIP1	AC	2-9	U-3	4	1.0	0.1	0.9	1.1
IIIP2	BD	2-9	A One-way	12	11.2	1.1	9.7	12.6
IIIP1′	AC	2-9	U-1	6	0.7	0.3	0.3	1.0
Willa B. Player Hall								
IVF1	A	1	U-2	16	7.7	0.7	6.9	8.6
IVF2	BC	1	U-2	16	8.3	0.6	7.5	9.1
IVF3	A	2	U-2	12	4.9	0.5	4.2	5.5
IVF4	BCD	2	U-2	12	5.2	0.3	4.8	5.7
IVF5	A	3	U-2	12	3.7	0.2	3.4	4.0
IVF6	BCD	3	U-2	12	3.7	0.2	3.4	3.9
IVF2′	ABCD	1	U-1	8	1.9	0.1	1.8	2.0
IVF4′	ABCD	2	U-1	8	1.3	0.03	1.24	1.32
IVF6′	ABCD	3	U-1	8	0.5	0.04	0.43	0.52
IVP1	A	1	U-2	16	7.7	0.7	6.7	8.6
IVP2	BC	1	U-2	16	8.3	0.7	7.4	9.3
IVP3	A	2	U-2	12	4.6	0.5	4.0	5.2
IVP4	BCD	2	U-2	12	5.0	0.5	4.4	5.6
IVP5	A	3	U-2	12	3.1	0.2	2.8	3.3
IVP6	BCD	3	U-2	12	3.3	0.1	3.1	3.5
IVP3′	A	1-3	U-1	8	0.2	0.1	0.1	0.3
IVP4′	ABCD	1-3	A One-way	8	4.6	2.4	1.5	7.7

Table 8 (concluded)

Case*	Location: Side	Location: Story	Wall Type†	Wall Thick. (in.)	Predicted Collapse Overpressure, psi: Mean	Standard Deviation	10 Percent Probability Value	90 Percent Probability Value
North Carolina National Bank								
VF1	A	1	A One-way	13	3.9	0.7	3.0	4.8
VF2	B	1	A One-way	13	1.8	0.2	1.6	2.0
VF3	ABCD	2-8	A One-way	13	12.4	2.6	9.0	15.7
VP1	A	1	A One-way	17	16.4	4.2	11.0	21.8
VP2	B	1	A One-way	17	5.4	0.7	4.6	6.3
VP3	ABCD	2-8	A Two-way	13	15.7	4.0	10.5	20.8

* The prefix F identifies walls analyzed using field survey data, and P those analyzed using building plan data. The prime identifies interior partitions.

† Each wall is designated with a letter to identify the wall type and a number to identify the wall support condition. The key to the wall types and support cases are given in Table S-2

Table 9

WALL TYPE AND SUPPORT KEY

Letter	Wall Type
U	Unreinforced masonry unit wall
A	Arching wall
RC	Reinforced concrete wall

Number	Support Case
1	Two-way, simply supported on four edges
2	Two-way, fixed on four edges
3	Two-way, fixed on vertical edges and simply supported on horizontal edges
4	Two-way, simply supported on vertical edges and fixed on horizontal edges
5	One-way, simply supported on opposite edges
6	One-way, fixed on opposite edges
7	One-way, propped cantilever
8	One-way, cantilever

In direct contrast to the 19 percent maximum variation in the predicted collapse values for the exterior walls of Player Hall for the two sets of data, the collapse overpressures obtained from the analysis of the interior corridor partitions with the plan data can be seen from Table 8 to range from 2.4 to 9.2 times those obtained with the survey data. This large difference results primarily from the difference in the support conditions used in the two analyses. The interior corridor partitions were classified in the survey as load-bearing walls but were shown on the plans as inset between the floor beams of the interior structural frame. Therefore, for the analysis with the survey data, the wall resistance was assumed to be dependent on the flexural strength of the wall and the vertical in-plane forces on the wall. Because of these assumptions the predicted collapse overpressure varied from 1.9 psi for the first story to 0.5 psi for the third story. For the plan data analysis, the wall resistance was assumed to be developed by arching between the floor beams and the predicted collapse overpressure was found to be 4.6 psi for all story levels.*

Another interesting example of the effect of both the support conditions and the wall thickness on the collapse overpressure is shown by the results of the analysis of the Laura Cone Dormitory. The exterior walls on the longitudinal sides of the building were described in the survey information as having a 6-in. thick concrete block backing wythe, whereas the plans showed only a 4-in. thick concrete block backing.

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* As discussed in Section II, it was assumed in the survey data analysis that the interior load-bearing partitions between rooms had the same collapse values as the corridor partitions. However, the plans showed that the walls between rooms were of the nonload-bearing type without arching. A comparison of the plan data analysis for this wall (Case IVP3′) with the survey data analysis cases, shows approximately the same ratios, but in reverse, as those found for the corridor partitions.

Also, for the survey data analysis, the walls were assumed to develop arching between frame members. However, the plans indicated that the walls were not in direct contact with the spandrel beams and therefore the support conditions restricted the wall to a bending mode. As noted for Cases IIIF1 and IIIP1 in Table 8, these two factors resulted in a predicted collapse overpressure for the survey data analysis that was 7.6 times that for the plan data analysis. This difference was the largest between the survey and plan data analyses for any of the exterior walls of the five Greensboro buildings. In addition, for the walls on the transverse sides of the building (Cases IIIF2 and IIIP2), the difference in wall thickness used in the analyses was primarily responsible for the collapse overpressure determined from the plan data analysis exceeding that from the survey data analysis by a factor of over two.

Finally, an example of the effect of the wall thickness and the type of wall materials on the predicted collapse overpressure is demonstrated by the analysis of the North Carolina National Bank. The first story wall of the Bank was described in the survey data as 13-in. thick with a stone veneer and an 8-in. thick tile backing wythe. However, the plans showed that the first story wall was 17-in. thick and that the stone facing was backed with brick. The result of the analysis showed that the plan data indicated a wall collapse strength that was over four times that for the survey data.

Appendix

FIELD SURVEY DATA

By

M. D. Wright
Research Triangle Institute

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DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelters

A. Building Identification and Geometry

1. Building Name and Address Southern Furniture Exhibition Building, Green ST, High Point, N.C.

2. Standard Location 3541-00560 3. Facility Number ______

4. Number of Stories 11+B+SB 5. Height of Building 153'

6. Story Height: Sub Bas. 16'3" Bas. 10'4" 1st 19' Upper 12' (19' on 08)

Upper (If Change) 20' Story of Change 11

7. Dimensions: Side A 147 Side B 224

8. Plan Areas: a. Basement Sub B 14,000 15,000 b. First Story 28,000

c. Upper Stories 31000 d. Upper Stories if Change ___

9. Fallout Shelter Data:

Story	No. of Rooms with Shelter	Shelter Area	No. of Spaces
-1	3	14,000	1400
CC	2	14,800	1480
01	5	1300	130
02	1	20,610	2065
03-08	1/ea sty	155000	15,500

10. a. Plans Available Yes b. Specs. Available No

c. Location RTH d. Contact ______

11. Building Use [redacted]-59 12. Year Constructed 1962

13. Building Code Reference ______

14. General Condition Good

15. Hazards: none

Note A: Basement and Sub-basement appear to be re-enforced concrete with upper stories being steel frame.

oversize brick use metal bonding strips only.

wall insert should be cemented in - ie fill space below beam

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B. Structural Details

	Side A	Side B	Side C	Side D	Source
1. Type of ~~S~~ Sub- Structure	2(12")	2(12")	2(12")	2(12")	meas
2. Basement Exposure	10'	3'	0	0	"

3. Type of Exterior Walls:

[redacted]

		Type/ Thickness	Type/ Thickness	Type/ Thickness	Type/ Thickness	
Basement (open)	Wall	54/8	/	/	/	est
	Veneer	71/4	/	/	/	est
First	Wall	54/8	Typical	/	/	est
	Veneer	71/4	/	/	/	est
Upper	Wall	54/8	/	/	/	est
	Veneer	71/4	/	/	/	est
Upper (If Change)	Wall	/	/	/	/	
	Veneer	/	/	/	/	
Story of Change						

Wall Panel Dimensions Width 21' Height 12'

Support Conditions: 2

Cavity Wall (estimate width) in. 1"

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner ______ outer ______ N/A

Horizontal: inner ______ outer ______

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel ______

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel ______

Compressive Strength of Concrete ______

b. Masonry Walls:

Compressive Strength of Mortar ______ N/A

4. Aperture Data:						
	Basement(WxH)	110'x8'	0	0	0	meas
	Sill Ht.	0	0	0	0	meas
	First(WxH)	110'x16'	145'x16'	0	5'x16'	meas
	Sill Ht.	0	0	0	0	meas

	Side A	Side B	Side C	Side D	Source
Upper (WxH)	0	0	0	5'x16	meas
Sill Ht.	0	0	0	0	meas
Upper (If Change) (WxH)	147x20'	224x20'			meas
Sill Ht.	0	0			meas
Story of Change	11	11			

(no way to know without soil tests)

(See note A)

5.b Type of Foundation 120 — est.

6. Type of Frame 111 Sub Bsmt, Bsmt, 210 above — est

Dimensions of Columns Sub Bsmt 24"x24"; Bsmt up 12-10"WF — est

Dimensions of Beams Sub Bsmt 2' wide, 24" deep; Bsmt up 16"I — est

a. Reinforced Concrete Frame

Bar Size and Spacing — — N/A

Type Reinforcement — —

Concrete Compressive Strength — —

b. Steel Frames

Type of Steel — — N/A

Fireproofing for Steel Frames 6 (steel covered with [illegible] fireproofing) — est

c. Drop panel data: W. — L. — T. — — est.

7. Roof: Slope 12 Frame 25 Deck 33-3" Covering 42 — obs.

Height of Parapet Walls: Side: A. — B. — C. — D. 3' — meas

8. Floors: First (Bsmt) Frame 14 Deck 35 (8") (25 (10")) — obs. & est.

Upper Frame 13 Deck 23 (4") — obs. & est.

Upper (If Change) Frame — Deck — —

Story of Change — —

(Mr. H: 4" floor probably)

Framing into Bearing Walls: — —

Spans: Parallel to Side "A" 21' Parallel to Side "B" 21'

a. Reinforced Concrete Floors

Bar Size and Spacing — N/A

Type Reinforcement —

Concrete Compressive Strength —

b. Structural Steel Floors

Beam Size 16" I + 12" O.W.J. — est.

* 9. Type of Interior Partitions: Basement (Sub bsmt) 24 (8") — obs.

First 24 (8") + 28 — obs.

Upper ([redacted]) 28 — obs.

Story of Change — —

* No significant interior partitions on upper stories except around stairs and elevators

		Source
C.	**Geological Data**	
	1. Depth of Water Table ______ 2. Rock Below Grade ______	N/A
	3. Soil Type ______	
	4. Design Bearing Capacity of Soil ______	

D. **Fire Vulnerability**

		Side A	Side B	Side C	Side D	Source
1. Adjacent Buildings -	Stories	–	–	10	–	Obs.
	Distance	–	–	0	–	"
	Type of Construction	–	–	NC	–	"

2. Velocity and Direction of Prevailing Winds ______ N/A

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Identify Side A on all floor plan sketches. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail if available. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods if such information is available.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address Greensboro Public Library
Greene and Gaston Sts. Greensboro, N.C.

2. Standard Location 3541.0009 3. Facility Number 00435

4. Number of Stories 2(B+SB) 5. Height of Building 30'

6. Story Height: Bas. 11'8" Sub bas. 7'7" 1st 15'2" Upper 15'2"

Upper (If Change) — Story of Change —

7. Dimensions: Side A 143 Side B 139

8. Plan Area: a. ~~Basement~~ Subbasement 19877 b. First Story 17346

c. Upper Stories 19170 d. Upper Stories if Change —

9. Fallout Shelter Data:

Story	No. of Rooms with Shelter	Shelter Area	No. of Spaces
-1	2	19860	1986
0	2	19860	1986

10. a. Plans Available yes b. Specs. Available no

c. Location RTI d. Contact

11. Building Use 26 12. Year Constructed 1964

13. Building Code Reference

14. General Condition good

15. Hazards: none

B. Structural Details

		Side A	Side B	Side C	Side D	Source
1.	Type of ~~Structure~~ sub structure	2(12")	2(12")	2(12")	2(12")	obs. est.
2.	Basement Exposure	0	0	0	0	obs.
3.	Type of Exterior Walls:					

		Type/ Thickness	Type/ Thickness	Type/ Thickness	Type/ Thickness	
Basement	Wall	–/–	–/–	–/–	–/–	–
	Veneer	–/–	–/–	–/–	–/–	–
First	Wall	54/8"	54/8"	54/8"	54/8"	meas. est. obs
	Veneer	72/4"	72/4"	71/4"	71/4"	"
Upper	Wall	54/8"	54/8"	54/8"	54/8"	"
	Veneer	72/4"	72/4"	71/4"	71/4"	
Upper (If Change)	Wall	–/–	–/–	–/–	–/–	–
	Veneer	–/–	–/–	–/–	–/–	–
Story of Change		–	–	–	–	–

exterior wall veneer on sides A + B is mosaic cast stone panels.

Wall Panel Dimensions Width 25' Height 15'

Support Conditions: 2 — est.

Cavity Wall (estimate width) in. none

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner N.A. outer N.A. —

Horizontal: inner N.A. outer N.A. —

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel N.A. —

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel N.A. —

Compressive Strength of Concrete N.A. —

b. Masonry Walls:

Compressive Strength of Mortar N.A. —

4. Aperture Data:						
	Basement(WxH)	–	–	–	–	–
	Sill Ht.	–	–	–	–	–
	First(WxH) *	40'x6'	30'x6'	–	–	meas.
	Sill Ht.	38"	38"	–	–	"

Large apertures at stairways on side A + B are decorative only. There is a wall inside them. See 1st story sketch.

*Apertures in A-B corner extend from ground level to the roof. 32' along side A and 19' along side B.

	Side A	Side B	Side C	Side D	Source
Upper (WxH) *		2'x6'	—	—	obs
Sill Ht.		30"	—	—	
Upper (If Change) (WxH)	—	—	—	—	—
Sill Ht.	—	—	—	—	—
Story of Change	—	—	—	—	—

5. Type of Foundation 120 — est.

6. Type of Frame 111 — obs.

Dimensions of Columns 22"x22" 18"x18" 14"x14" — meas.

Dimensions of Beams 20" wide x 12" deep, Beams over columns; 40" wide x 30" deep, girders over columns — meas.

a. Reinforced Concrete Frame

Bar Size and Spacing N. A. —

Type Reinforcement N. A. —

Concrete Compressive Strength N. A. —

b. Steel Frames

Type of Steel — —

Fireproofing for Steel Frames — —

c. Drop panel data: W. — L. — T. —

roof is 2-way ribbed joists over auditorium

7. Roof: Slope 12 Frame 26 Deck 35 / 36 (4") Covering 42 — est. / obs.

Height of Parapet Walls: Side: A. 36" B. 36" C. 36" D. 36" — meas.

8. Floors: First Frame 14 Deck 25 (4") — obs. est

Basement floor has no beams. It consists of a 4" thick reinforced concrete slab supported by the columns.

Upper Frame 14 Deck 25 (4") — obs. est.

Upper (If Change) Frame — Deck — —

Story of Change — —

Framing into Bearing Walls: — —

Spans: Parallel to Side "A" 25' 50' in auditorium on Story 02. Parallel to Side "B" 25' — meas.

a. Reinforced Concrete Floors

Bar Size and Spacing N. A. —

Type Reinforcement N. A. —

Concrete Compressive Strength N. A. —

b. Structural Steel Floors

Beam Size — —

9. Type of Interior Partitions: Subbasement ~~Basement~~ 26 (12") — meas.

First 27 (6") + 24 (8") — meas. est.

Upper ~~(If Change)~~ 24 (8") 27 (6") — est. meas.

Story of Change 50% 50%

		Source
C.	Geological Data	
	1. Depth of Water Table ________ 2. Rock Below Grade ________	N/A
	3. Soil Type ________	N/A
	4. Design Bearing Capacity of Soil ________	N/A
D.	Fire Vulnerability	

	Side A	Side B	Side C	Side D	Source
1. Adjacent Buildings - Stories	–	5	2	2	obs
Distance	–	72'	0	0	obs.
Type of Construction		NC	NC	NC	obs

2. Velocity and Direction of Prevailing Winds ________ ________

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Identify Side A on all floor plan sketches. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail if available. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods if such information is available.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address Laura Cone Dorm
West Market Street, UNC-G, Greensboro N.C.

2. Standard Location 3541-0008 0 3. Facility Number 00384

4. Number of Stories 9 + BSMT 5. Height of Building 86'

6. Story Height: Bas. 11'4" 1st 11'5" Upper 9'4"
Upper (If Change) — Story of Change —

7. Dimensions: Side A 193 Side B 39

8. Plan Area: a. Basement 5600 b. First Story 8100
c. Upper Stories 8100 d. Upper Stories if Change —

9. Fallout Shelter Data:

Story	No. of Rooms with Shelter	Shelter Area	No. of Spaces
03	32	3630	363
04	32	3630	363
05	32	3630	363
06	32	3630	363
07	32	3630	363
08	1 (hall)	1040	104

10. a. Plans Available yes b. Specs. Available no
c. Location RTI d. Contact

11. Building Use Dormitory (12) 12. Year Constructed 1967

13. Building Code Reference

14. General Condition very good

15. Hazards: none

B. <u>Structural Details</u>

		Side A	Side B	Side C	Side D	Source
1.	Type of Sub-Structure	2-12"	2-12"	1	1	est
2.	Basement Exposure	100%/0	0	100%	100%	meas
3.	Type of Exterior Walls:					

		Type/ Thickness	Type/ Thickness	Type/ Thickness	Type/ Thickness	
Basement	Wall	54 / 6	– / –	54 / 8	54 / 8	meas
	Veneer	71 / 4	– / –	71 / 4	71 / 4	"
First	Wall	54 / 6	54 / 6	54 / 8	54 / 8	"
	Veneer	71 / 4	71 / 4	71 / 4	71 / 4	"
Upper	Wall	54 / 6	54 / 6	64 / 6	54 / 6	"
	Veneer	71 / 4	71 / 4	71 / 4	71 / 4	"
Upper (If Change)	Wall	– / –	– / –	/	/	–
	Veneer	– / –	/	/	– / –	
Story of Change		–	–	–	–	–

Wall Panel Dimensions Width 24' Height 7½'

Support Conditions: 2 est

Cavity Wall (estimate width) in. 1"

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner – outer – –

Horizontal: inner – outer – –

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel – –

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel – –

Compressive Strength of Concrete – –

b. Masonry Walls:

Compressive Strength of Mortar N/A –

		Side A	Side B	Side C	Side D	Source
4. Aperture Data:	Basement (WxH)	5'1"x9' | 5'1"x6'6"	—	6'4"x2'4" | 8'x10'	18'x9'	meas
	Sill Ht.	0 | 2'	—	6'8" | 4'	0	"
	First (WxH)	5'6"x8' | 23'x7'	26'x9'	6'6"	18'x9'	"
	Sill Ht.	10" | 2'	6'	3'	6	"

See sketch for exact placement

	Side A	Side B	Side C	Side D	Source
Upper (WxH)	6'8"x5'1"	→	→	→ typical	[illegible]
Sill Ht.	2'7"	→	→	→	meas
Upper (If Change) (WxH)	—	—	—	—	—
Sill Ht.	—	—	—	—	—
Story of Change	—	—	—	—	—

5. Type of Foundation 122 — est.

6. Type of Frame 210

typical estimated as 14" WF columns and 12" I Beams

Dimensions of Columns 24 [illegible] / 15" BSMT 15" upper — meas.

Dimensions of Beams 12" I — meas

a. Reinforced Concrete Frame

Bar Size and Spacing — —

Type Reinforcement — —

Concrete Compressive Strength — —

b. Steel Frames

Type of Steel N/A —

Fireproofing for Steel Frames concrete (4) — est

c. Drop panel data: W. — L. — T. —

Frame 12" high corrugated deck on top of OWJ Frame

7. Roof: Slope 12 Frame 25 Deck 33(4") Covering 42 — est.

Height of Parapet Walls: Side: A. 3' B. 3' C. 3' D. 3' — est.

8. Floors: First Frame 12/15 Deck 23(4) — meas/est

Upper Frame 12/13 Deck 23(4) — meas/est

Upper (If Change) Frame — Deck — —

Story of Change — —

Framing into Bearing Walls: N/A —

Spans: Parallel to Side "A" 24' Parallel to Side "B" 21'10" — meas

a. Reinforced Concrete Floors

Bar Size and Spacing — N/A

Type Reinforcement — N/A

Concrete Compressive Strength — N/A

b. Structural Steel Floors

Beam Size 12" I and O.W.J.

9. Type of Interior Partitions: Basement 24 (6") — est

First 24 (6") — est

Corridor walls all load-bearing → Upper ~~[illegible]~~ 14+24 (6") — est

Story of Change — —

C. <u>Geological Data</u>

		Source
1.	Depth of Water Table ______ 2. Rock Below Grade ______	N/A
3.	Soil Type ______	N/A
4.	Design Bearing Capacity of Soil ______	N/A

D. <u>Fire Vulnerability</u>

		Side A	Side B	Side C	Side D	
1. Adjacent Buildings -	Stories	—	9	—	—	[illegible]
	Distance	—	110	—	—	''
	Type of Construction	—	NC	—	—	''
2. Velocity and Direction of Prevailing Winds ______						N/A

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Identify Side A on all floor plan sketches. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail if available. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods if such information is available.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry (WILLA B. PLAYER HALL)

1. Building Name and Address NEW DORMITORY BENNETT COLLEGE GREENSBORO, N.C.

2. Standard Location 3541 0013 3. Facility Number 00610

4. Number of Stories 3 5. Height of Building 37'

6. Story Height: Bas. — 1st 12'3" Upper 11' 3"

Upper (If Change) — Story of Change —

7. Dimensions: Side A 206 Side B 65

8. Plan Area: a. Basement 12,300 b. First Story 12,300

c. Upper Stories 12300 d. Upper Stories if Change —

9. Fallout Shelter Data:

Story	No. of Rooms with Shelter	Shelter Area	No. of Spaces
01	27	8910	891
02	8	2410	241

10. a. Plans Available yes b. Specs. Available NO

c. Location RTI d. Contact

11. Building Use 12 12. Year Constructed 1966

13. Building Code Reference

14. General Condition Good

15. Hazards: none

B. Structural Details

		Side A	Side B	Side C	Side D	Source
1.	Type of ~~Structure~~ Sub-structure	5(16")	5(16")	5(16")	5(16")	est.
2.	~~Basement~~ 1st story Exposure	2'/100%	100%	100% including areaway	2'	meas.
3.	Type of Exterior Walls:					

		Type/ Thickness	Type/ Thickness	Type/ Thickness	Type/ Thickness	Source
Basement	Wall	– / –	– / –	– / –	– / –	–
	Veneer	– /	– /	– /	– /	–
First	Wall	26/16"	26/16"	26/16"	26/16"	meas
	Veneer	– / –	– /	– /	– /	
Upper	Wall	26/12"	26/12"	26/12"	26/12"	meas
	Veneer	– / –	– /	– /	– /	
Upper (If Change)	Wall	– / –	– / –	– / –	– / –	–
	Veneer	– / –	– /	– /	– /	–
Story of Change		–	–	–	–	–

Wall Panel Dimensions Width ~~18'~~ 40', 12' Height ~~12'~~ 12'

Support Conditions: Bearing Wall (Vaulted load) obs

Cavity Wall (estimate width) in. none

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner – outer – –

Horizontal: inner – outer – –

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel – –

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel – –

Compressive Strength of Concrete – –

b. Masonry Walls:

Compressive Strength of Mortar Not available –

4.	Aperture Data:					Source
	Basement(WxH)	–	–	–	–	–
	Sill Ht.	–	–	–	–	–
	First(WxH)	44"x61"	44"x61"	44"x61"	44"x61"	meas
	Sill Ht.	32"	32"	32"	32"	meas

Doorways not included
See photographs.

	Side A	Side B	Side C	Side D	Source
Upper (WxH)	44"x61"	44"x61"	44"x61"	44"x61"	meas.
Sill Ht.	32"	32"	32"	32"	meas.
Upper (If Change) (WxH)	—	—	—	—	—
Sill Ht.	—	—	—	—	—
Story of Change	—	—	—	—	—

		Source
5. Type of Foundation	110 + 120	est.
6. Type of Frame	500 + 210	obs.
Dimensions of Columns	8" 1st 6" upper (interior columns)	meas. est.
Dimensions of Beams	12" I + open web joists	obs.
a. Reinforced Concrete Frame		
Bar Size and Spacing	—	—
Type Reinforcement	—	—
Concrete Compressive Strength	—	—
b. Steel Frames		
Type of Steel	not available	—
Fireproofing for Steel Frames	6" + concrete block	obs.
c. Drop panel data: W. — L. — T. —		
7. Roof: Slope 11 Frame 23 Deck 38 Covering 45		obs.
Height of Parapet Walls: Side: A. — B. — C. — D. —		—
8. Floors: First Frame — Deck —		—
Upper Frame 12 + 13 Deck 23 (3½")		obs., est.
Upper (If Change) Frame — Deck —		—
Story of Change	—	—
Framing into Bearing Walls:	joists extend into brick.	obs.
Spans: Parallel to Side "A" 12' Parallel to Side "B" 20' (max)		meas
a. Reinforced Concrete Floors		
Bar Size and Spacing	not available	—
Type Reinforcement	" "	—
Concrete Compressive Strength	" "	—
b. Structural Steel Floors		
Beam Size	12" I + open web joists	est.
9. Type of Interior Partitions: Basement	—	—
First	14 (8")	meas
Upper ([redacted])	14 (8")	meas
Story of Change	—	—

joists estimated to be supported on a metal plate and tied with an anchor bar.

						Source
C.	Geological Data					
	1. Depth of Water Table ______ 2. Rock Below Grade ______					N/A
	3. Soil Type ______					N/A
	4. Design Bearing Capacity of Soil ______					N/A
D.	Fire Vulnerability					
		Side A	Side B	Side C	Side D	
	1. Adjacent Buildings - Stories	—	—	—	2	obs
	Distance	—	—	—	45'	obs.
	Type of Construction	—	—	—	NC	obs
	2. Velocity and Direction of Prevailing Winds				—	N/A

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Identify Side A on all floor plan sketches. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail if available. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods if such information is available.

See shelter marking sketches for floor plans. Story 03 is the same as story 02 except that the large room in the center is partitioned into student rooms.

DATA COLLECTION FORM

Structural Characteristics for All-Effects Shelter

A. Building Identification and Geometry

1. Building Name and Address North Carolina National Bank S. Main St. High Point, N.C.

2. Standard Location 3541-0058 3. Facility Number 02852

4. Number of Stories 8+B+mezz 5. Height of Building 103

6. Story Height: Bas. 13' 6" 1st 15' mezz. 15' Upper 10' 6"

Upper (If Change) — Story of Change —

7. Dimensions: Side A 50 Side B 115

8. Plan Area: a. Basement 7245 b. First Story 5750

c. Upper Stories 5750 d. Upper Stories if Change —

9. Fallout Shelter Data:

Story	No. of Rooms with Shelter	Shelter Area	No. of Spaces
0	6	6474	168
1	1	2460	246
2	1 (corridor)	820	82
3,4,5,6	17/story	2630/story	263/story
7	1 (corridor)	820	82

10. a. Plans Available yes b. Specs. Available no

c. Location Q.T.F. d. Contact

11. Building Use 55 12. Year Constructed 1922

13. Building Code Reference

14. General Condition good

15. Hazards: none

B. Structural Details

	Side A	Side B	Side C	Side D	Source
1. Type of all Structure	2(12")	2(12")	2(12")	2(12")	2(12")
2. Basement Exposure	0	avg. 2'	3'	—	[illegible]

3. Type of Exterior Walls:

		Type/ Thickness	Type/ Thickness	Type/ Thickness	Type/ Thickness	
Basement	Wall	51/8"	51/8"	51/8"	51/8"	meas
	Veneer	71/4"	71/4"	71/4"	71/4"	est
First	Wall	59/8"	59/8"	59/8"	59/8"	meas
	Veneer	71/4"	71/4"	71/4"	71/4"	est
Upper	Wall	59/8"	59/8"	59/8"	59/8"	[illegible]
	Veneer	71/4"	71/4"	71/4"	71/4"	[illegible]
Upper (If Change)	Wall	—/—	—/—	—/—	—/—	—
	Veneer	—/—	—/—	—/—	—/—	—
Story of Change		—	—	—	—	—

Exterior walls measure approximately 13"

Exposed basement walls on side B and 1st story walls below the windows on A + B are faced with Marble. Remainder of 1st story walls on A + B are faced with cast stone panels.

Wall Panel Dimensions Width 15 / 15 Height 10½ / 30

Support Conditions: 2 — est

Cavity Wall (estimate width) in. none

a. Reinforced Concrete Walls:

Bar Size and Spacing:

Vertical: inner N.A. outer N.A. —

Horizontal: inner N.A. outer N.A. —

Distance From Outer Wall Surface to Centroid of Outer Layer of Steel N.A. —

Distance From Outer Wall Surface to Centroid of Inner Layer of Steel N.A. —

Compressive Strength of Concrete N.A. —

b. Masonry Walls:

Compressive Strength of Mortar N.A. —

4. Aperture Data:						
	Basement (WxH)	—	—	—	—	—
	Sill Ht.					
	First (WxH)	12'x25'	(5'x24') 10'x24'	5'x8'	—	meas
	Sill Ht.	0	(3½')	3½'	3½'	-	meas

Some of the large 1st story apertures on side B from [illegible] covered with cast stone panels. (see photos)

Some of the apertures on upper stories have been bricked up. (see photos; sides C + D

	Side A	Side B	Side C	Side D	Source
Upper (WxH)	43"x67"	43"x67"	43"x67"	43"x67"	meas.
Sill Ht.	30"	30"	30"	30"	meas.
Upper (If Change) (WxH)	—	—	—	—	—
Sill Ht.	—	—	—	—	—
Story of Change	—	—	—	—	—

5. Type of Foundation 120 — est.

6. Type of Frame 210 (steel columns + Beams encased in reinforced concrete) — est.

Dimensions of Columns meas (12"x12") on base. est. 6 WF — meas + est

Dimensions of Beams 6" I — est.

a. Reinforced Concrete Frame

Bar Size and Spacing — —

Type Reinforcement — —

Concrete Compressive Strength — —

b. Steel Frames

Type of Steel N.A.

Fireproofing for Steel Frames 4(3") — est.

c. Drop panel data: W. — L. — T. —

floors and roof are all 2-way ribbed joists with tile inserts.

7. Roof: Slope 12 Frame 23 Deck 31(3") Covering 41 — est.

Height of Parapet Walls: Side: A. 45" B. 45" C. 45" D. 45" — meas.

8. Floors: First Frame 12 Deck 26 (4") — est. obs

Upper Frame 12 Deck 26 (4") — "

Upper (If Change) Frame — Deck — —

Story of Change — —

Framing into Bearing Walls: — —

Spans: Parallel to Side "A" 15' Parallel to Side "B" 15' — meas.

a. Reinforced Concrete Floors

Bar Size and Spacing N.A. — —

Type Reinforcement N.A. — —

Concrete Compressive Strength N.A. — —

b. Structural Steel Floors

Beam Size 6" I encased in Reinforced Concrete. — est.

9. Type of Interior Partitions: Basement 21 (12") — est.

First 29 (6") — est.

Upper ~~(If Change)~~ 30(6")+29(3") — est.

Story of Change — —

Interior partitions on the 2nd through 8th floors were built with numerous apertures extending from approximately 4½' above the floor to the ceiling. These have been filled in with very light wood panels

Partitions are 6" around stairs and elevators and 4" in corridor + between rooms.

						Source
C.	Geological Data					
	1. Depth of Water Table ________ 2. Rock Below Grade ________					N/A
	3. Soil Type ________					N/A
	4. Design Bearing Capacity of Soil ________					N/A
D.	Fire Vulnerability					
		Side A	Side B	Side C	Side D	
	1. Adjacent Buildings - Stories	—	9	—	2	obs.
	Distance	—	75'	—	0	obs.
	Type of Construction	—	RC	—	RC	obs.
	2. Velocity and Direction of Prevailing Winds ________					N/A

E. Provide sketches of basement, first floor, and upper floors showing partition locations and floor openings. Identify Side A on all floor plan sketches. Provide sketches or photographs of all four exterior walls showing location of apertures. Provide sketch of exterior wall detail if available. For reinforced concrete floors and frame, provide sketch of floor detail and column detail, showing location of reinforcing rods if such information is available.

REFERENCES

1. Wiehle, C. K., and J. L. Bockholt, Existing Structures Evaluation, Part I: Walls, Stanford Research Institute (for Office of Civil Defense), Menlo Park, California, November 1968 (AD-687 293).

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NOMENCLATURE

E_c	Modulus of elasticity of concrete, psi
E_m	Modulus of elasticity of masonry, psi
f'_c	Compressive strength in concrete, psi
f'_m	Ultimate compressive strength of masonry unit wall, psi
f_r	Modulus of rupture of masonry, psi
L_H	Horizontal length (width) of wall, in.
L_V	Vertical length (height) of wall, in.
P_v	Total vertical force per unit width, lb/in.
S	Clearing distance, ft
t_f	Thickness of flange of hollow masonry block unit, in.
t_w	Thickness of wall, in.
γ	Unit weight, pcf